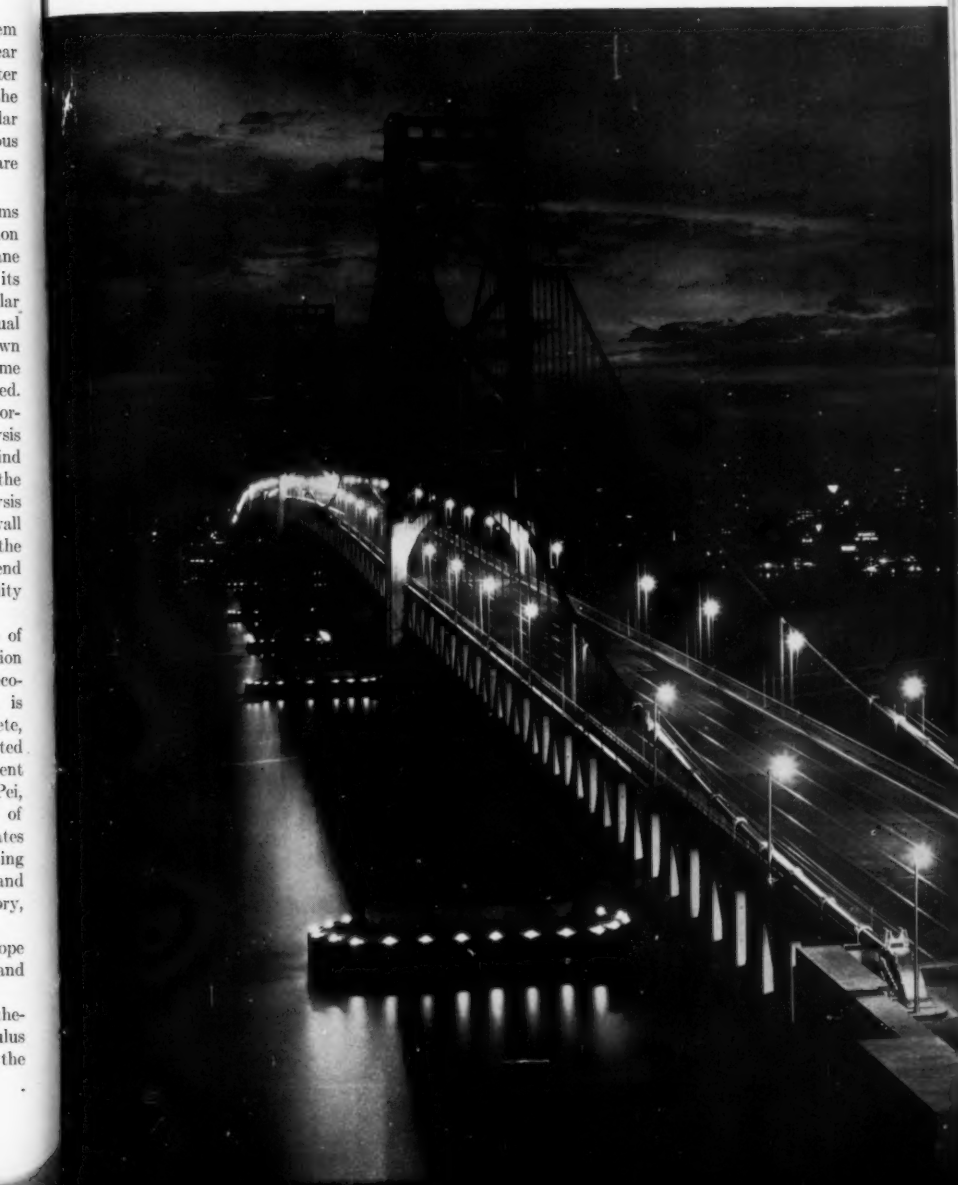
ACI News Letter

Vol. 22 No. 6 JOURNAL of the AMERICAN CONCRETE INSTITUTE February 1951



84% of the 279 projects undertaken by Intrusion-Prepakt during 1950 were done for "repeat" customers. These jobs were executed on a great variety of projects, including 125 railroad and highway bridges, 40 dams and locks, 10 tunnels, 61 foundations and piles, and 43 other PREPAKT installations and services.

The superior resistance to deterioration of PREPAKT CONCRETE plus the many advantages in placing it in "hard-to-get-at" installations have proved to users that Intrusion-Prepakt provides worth-while economies.

Prepakt representatives will gladly describe the advantages of Intrusion-Prepakt and the engineering and field construction services available. Call or write the office nearest you.

PREPAKT MAINTAINS A QUALIFIED DESIGN AND SUPERVISORY SERVICE PLUS A COMPLETE CONSTRUCTION ORGANIZATION

CONTRACTORS
ENGINEERS

PREPAKT

SPECIAL
SERVICES

INTRUSION-PREPAKT, INC. THE PREPAKT CONCRETE CO.

CHICAGO • TORONTO CLEVELAND 14, OHIO SEATTLE • PHILADELPHIA

ZURICH - PARIS - MADRID - STOCKHOLM - HELSINKI - WIESBADEN

ACI

News Letter Contents

February 1951

On the Cover—The San Francisco-Oakland Bay Bridge from the Oakland side with the San Francisco skyline in the distance.

Courtesy San Francisco Convention and Tourist Bureau

Convention time table..... 3

So you're going to San Francisco?—Program highlights..... 4

The months ahead..... 5

Who's Who

- Walter H. Price.....14
- E. A. Whitehurst.....14
- H. F. Clemmer.....14
- Joseph J. Shideler, Harold W. Brewer and Wilbur H. Chamberlin.....14
- Otto Safir.....16
- M. Fomerod.....16

Honor Roll.....20

New Members.....24

Positions and Projects—ACI Members.....27

Tools, Materials, Services..34

Synopses.....35

Discussion.....41

List of Advertisers.....42

Time Table for 47th Annual Convention

February 20-22, 1951
St. Francis Hotel
San Francisco

Tuesday, February 20

9:00 a.m.—Registration begins
—Technical committee meetings

10:00 a.m.—First general convention session—Standards
—Colonial Ballroom

2:00 p.m.—Convention session—Design
—Colonial Ballroom

7:00 p.m.—Trip to Chinatown

7:30 p.m.—Technical committee meetings

Wednesday, February 21

(Concurrent sessions will be held morning and afternoon in the Colonial Ballroom and Green Room)

9:00 a.m.—Convention session—Materials and properties—Colonial Ballroom
—Convention session—Design—Green Room

12:15 p.m.—General Luncheon—Colonial Ballroom

2:30 p.m.—Convention session—Precast concrete—Colonial Ballroom
—Convention session—Construction—Green Room

7:00 p.m.—Dinner party

8:30 p.m.—Technical committee meetings

Thursday, February 22

9:00 a.m.—Open meeting of ACI Committee 115, Research—Colonial Ballroom

2:00 p.m.—Final convention session—Panel discussion
—Colonial Ballroom

5:30 p.m.—San Francisco Bay Cruise

So you're going to San Francisco?

Convention Program Highlights

Three days of "concrete" information will include sessions on design of concrete structures, materials and properties of concrete, concrete construction, precast concrete, a panel discussion of concrete problems and the annual open session on concrete and cement research. Synopses of all convention papers were published in the January *News Letter*.

Concurrent sessions, inaugurated last year at the Chicago convention, will be continued to allow a greater number of topics to be presented.

Revision of standards

ACI technical committees will present revisions of three ACI Standards—the Building Code, the detailing manual for reinforced concrete structures, and specifications for concrete pavements and bases. A new standard—recommended practice for the application of pneumatically placed mortar —will also be presented for adoption.

Proposed changes in "Building Code Requirements for Reinforced Concrete (ACI 318-47)" allow for the improved properties of new-style deformed reinforcing bars complying with ASTM Specification A 305-49 and the recommendations of ACI Committee 208 on bond stress.

It is proposed to decrease the allowable bond stresses in plain bars (including the old types of deformed bars) and increase the allowable bond stresses for the new types of bars over those previously allowed for the old types. Top bars, those having more than 12 in. of concrete under them, are assigned lower bond stresses than bars in other positions.

All plain bars must be hooked, which corresponds to special anchorage under the old provisions. The new bars develop sufficient anchorage by bond alone to correspond to special anchorage with the old type bars, resulting in less total steel required.

Proposed revisions of "Specifications for Concrete Pavements and Bases (ACI 617-44)" will include sections covering air entrainment, removal of forms, premolded joint fillers and joint filling materials, and method of placing reinforcement. Definitions have been added under soil foundation preparation and other parts of this section have been revised.

Proposed revisions of "Manual of Standard Practice for Detailing Reinforced Concrete Structures (ACI 315-48)" will bring the manual into conformity with proposed changes of the Building Code. Changes are also proposed in reinforcing bar designations to conform to the numbered designation of the U. S. Department of Commerce.

"Recommended Practice for the Application of Mortar by Pneumatic Pressure," by Committee 805 will be proposed as a new standard. It presents briefly the advantages and disadvantages of pneumatically-placed mortar and establishes recommended practices for placing and mixing shotcrete, qualifications and duties of workmen, preparation of surface before shotcreting, sequence of application, and other items involved in good shotcreting.

Precast concrete—protection from atomic blast

A special session devoted to precast concrete will feature a paper on its use as protection against atomic blast. Advances in precast floor systems and precast units and structural elements will be described, as well as precast concrete construction techniques.

Precast concrete can be used to provide protection against atomic blast, as A. Amirikian, Head Designing Engineer, Bureau of Yards and Docks, Dept. of the Navy, Washington, D. C., will explain. The technique is suitable for new construction as well as being adaptable to existing structures

Continued on p. 8

The months ahead

Last year I drove a considerable distance along a water front delayed for some hours by wisps of fog, with visions of the road reasonably clear one minute, blotted out the next in wraith-like ribbons of mist. The fog floated in waving bands at changing levels with respect to car and pavement. I saw a clear road, not much traveled and an instant later saw nothing with clarity.

The problems of motor car progress were much like those of ACI management—under increasing perplexities as a cold war gained in heat—at this moment of writing not yet become a hot war even while increasing numbers of men are drawn into it, some not to return to their peaceful ways of life.

ACI is gearing itself to a problematical situation—making, like many of you who read these lines, two, perhaps three sets of plans for the days, perhaps months, or years ahead.

Each set of plans is marked:

1—IF (so and so);

2—IF (such and such);

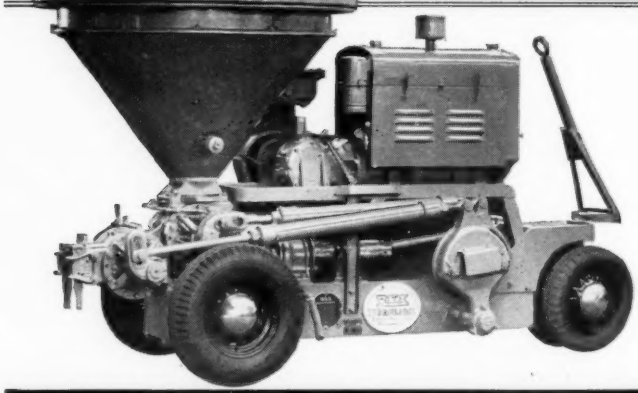
3—IF (some other eventuality).

Under these pressures of uncertainty, to what efforts shall ACI give precedence? Paper and printers' ink may be hard to get in adequate supply. What we publish will cost more per word, more in effort in the greater urgency of brevity—to pin down what is salient, shredded from its husks of the superfluous.

These lines express what I believe to be in the minds of every ACI staff member, every ACI Board member and Technical Activities Committee member—and (that's the idea of this page), every ACI Member, hoping that he will tell us on what activities he thinks we should concentrate.

Harry Kluppel

REX PUMPCRETE ®



YOUR AUTOMATIC INSPECTOR OF CONCRETE QUALITY

Rex Pumpcrete . . . *the pump that pumps concrete through a pipe line* . . . is an *automatic inspector* that inspects the quality of every batch of concrete pumped to the forms.

FIRST. Where segregation of the batch has occurred through mishandling or unsatisfactory mixing, the remixing hopper of the Pumpcrete will automatically correct this condition, assuring highest quality concrete.

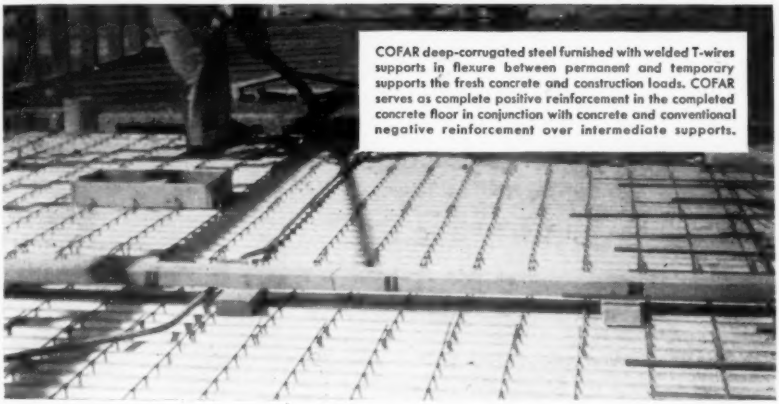
SECOND. When the batch contains inferior aggregates, or is a poor mix, the Pumpcrete will refuse to pump it! It *automatically* inspects batch quality!

THIRD. The piston and valves of the Pumpcrete form the plastic concrete into cylinders which are placed in the sealed pipe line and pumped directly to the forms. There is no chance for segregation of the concrete during travel through the *sealed* pipe line. Concrete arrives at the forms in the same unsegregated, high quality condition as it was when it entered the pipe line.

For highest quality concrete . . . for automatic inspection of concrete quality, rely on Rex Pumpcrete. For the complete story, write Chain Belt Company, 1713 W. Bruce Street, Milwaukee 4, Wis.

CHAIN BELT REX COMPANY CONSTRUCTION MACHINERY

COFAR COmbination Form And Reinforcement



COFAR deep-corrugated steel furnished with welded T-wires supports in flexure between permanent and temporary supports the fresh concrete and construction loads. COFAR serves as complete positive reinforcement in the completed concrete floor in conjunction with concrete and conventional negative reinforcement over intermediate supports.

STRONG

Guaranteed yield point (.1% offset) over 80,000 psi effective as reinforcement, balanced design, up to full tensile strength over 100,000 psi, independent of form stress.

VERSATILE

Minimum 24ga. —(A_S .35 sq. in. per foot width); maximum 18 ga. —(A_S .73 sq. in. per foot width); to meet full range concrete slab design. COFAR reinforcement satisfies all normal concrete slab design.

Cold-drawn high-strength T-wires not over 6″ c.c. are welded to the deep-corrugated steel, and constitute temperature reinforcement in the slab, mechanical anchorage and positive shear transfer from concrete to steel.

PERMANENT

COFAR sheets hot-dip galvanized, with specification heavy galvanizing, provide complete permanence for normal interior exposures. Concrete and steel permanently bonded by chemical union of calcium zincate for all normal loads. Fire resistance for any exposure by lightweight modern ceiling plaster or fibre protection.

ECONOMICAL

COFAR is concrete reinforcement, steel weight substantially equal to conventional reinforcement; *COFAR also is the concrete form.* Form sheeting as well as concrete runways are eliminated. COFAR comes detailed and cut to fit each job. Important material, labor, and time savings are made.

AVAILABLE

Advice for design engineers, review and estimates. COFAR is available — Granco Steel Products Company.

GRANCO STEEL PRODUCTS CO.
(*Subsidiary of* GRANITE CITY STEEL CO.)
GRANITE CITY, ILLINOIS

SEND FOR AIA FILE NO. 4E4

The Improved WASHINGTON

PRESS-UR-METER

NOW IN WORLD-WIDE USE

Universal acceptance, including several foreign countries, has established the PRESS-UR-METER *as the outstanding device for testing air entrained concrete*—faster than any other instrument on the market, with guaranteed accuracy.

HERE IS BIG NEWS!

SPECIFIC GRAVITY and MOISTURE DETERMINATIONS of aggregates may be quickly made using the NEW CHART now furnished with the PRESS-UR-METER. This Chart has been carefully prepared and numerous tests assure the accuracy and reliability of this method.

LET US TELL YOU about this important extra value of the PRESS-UR-METER for testing air entrained concrete and also designing concrete mixes.

For complete information, write to

CHARLES R. WATTS & CO.

4121 – 6th Ave. N. W. Seattle 7, Wash.

Exclusive sales agents

Program highlights

Continued from p. 4

by providing them with a protective shell. Mr. Amirikian will describe details of assembly and arrangement of precast framing elements to form buildings and shelters. He also plans to present an analysis of anticipated resistance of such structures to blast pressures.

Reinforced concrete floors and roofs for office and school buildings and apartments are being made of precast blocks assembled to form a joist or slab, and also of precast joists with precast filler block. The high cost of carpentry has brought about this successful attempt to eliminate most of the previously required formwork. Prof. F. N. Menefee, Dept. of Civil Engineering, University of Michigan, Ann Arbor, Mich., will present a paper describing the latest developments in precast floor units.

A. G. Streblow, President, Basalt Rock Co., Inc., Napa, Calif., will describe the manufacture and use of precast structural elements. Otto Safir, consulting engineer, Vancouver, B. C., Canada, will discuss the application of precast concrete to warehouse, parking garage and retaining wall construction.

With the increasing use of precast concrete, new construction techniques have emerged; one of these is tilt-up construction. F. Thomas Collins, consulting engineer, San Gabriel, Calif., will describe developments in this field, as well as recommended construction methods.

R/C structures resist quakes

Design sessions will feature papers on the design of concrete buildings to resist earthquakes, parking garages, splices for reinforcing bars and effect of lateral loads on arches.

Since earthquakes have been of concern to builders on the West Coast for many years and the possibilities of atomic blasts must now be considered by engineers and builders around the world, one design session will feature several papers on earthquake resistant design. The same reinforced concrete design and construction techniques can also be applied to structures to resist

hurricanes, atomic blasts or other disasters created by nature or man. The possibilities of atomic warfare call for a reorientation of construction practice—which is claimed to be economically feasible through the use of reinforced concrete.

John J. Gould, consulting engineer, San Francisco, Calif., will describe the design and construction of 13-story, reinforced concrete apartment buildings. These reinforced concrete bearing wall buildings were erected at low cost and are capable of resisting earthquakes, hurricanes and offer considerable protection against atomic blasts.

Prof. R. R. Martel, California Institute of Technology, Pasadena, Calif., is scheduled to discuss earthquake resistant design theory and what has been learned from observations of earthquake damage.

Charles S. Whitney, Boyd G. Anderson and Mario Salvadori, Ammann and Whitney, Consulting Engineers, New York, N. Y., will present a paper describing a new method for the design of structures under dynamic loading which they offer as applicable to earthquake design.

Prestressed concrete for highway construction

Several convention papers are aimed to serve the highway engineer and contractor; the topics include prestressed highway bridges, rigid pavement joint problems, finishing and curing methods for improving the durability of concrete surfaces, and resurfacing pavements.

Prestressed concrete construction may be the answer to shortages of cement and reinforcing and structural steel that are bound to face highway builders in the near future. Prestressed structures for freeways also promise savings in cost due to the reduced depth of span and resulting changes in design and construction. At the same time, prestressing would aid in the development of economical prefabricated designs for standard highway structures.

Stewart Mitchell, Principal Bridge Engineer, California Division of Highways, Sacramento, Calif., will discuss the use of prestressed concrete for highway bridges. He will devote a portion of the paper to California's experiences with prestressed concrete in the construction of a pedestrian bridge.

WHAT WILL CALCIUM CHLORIDE DO FOR GOOD CONCRETE?

HERE ARE THE FACTS

Here's a brand new, 40-page semi-technical book which clearly presents the facts on the use of Calcium Chloride. Filled with graphs, tables and charts, and developed through research conducted by nationally recognized authorities, the book contains much material not heretofore available. This information is of interest to contractors, architects, engineers, plant operators and men in the allied fields.

For example . . . DO YOU KNOW—

- What Calcium Chloride will do for cool and cold weather concrete?
- The effect of Calcium Chloride on slump?... flow? ... workability? ... density? ... water-cement ratio?
- The effect on setting time?...early strength?... ultimate strength?
- How much Calcium Chloride will reduce the curing period?
- The effect on volume change?...surface wear?

SOLVAY Calcium Chloride

These are just a few of the questions answered in this new book. "The Effects of Calcium Chloride on Portland Cement" is just off the press. Write for your copy today —no obligation.

SOLVAY SALES DIVISION,
Allied Chemical & Dye Corporation,
40 RECTOR STREET, NEW YORK 6, N. Y.

Please send my copy of the new, 40-page book, "The Effects of Calcium Chloride on Portland Cement".

☐ Contractor ☐ Architect
☐ Plant Operator ☐ Engineer

Name.....

Company.....

Address.....

City.....State..... JC-2

Francis Hveem, construction engineer, also with the California Division of Highways, will describe design and construction features of concrete pavements now being placed in California. A method of resurfacing concrete pavements with 1 in. of concrete will be described by H. Walter Hughes, Associate Engineer, Dept. of Public Works, Rochester, N. Y.

In line with highway departments' constant striving for durable concrete surfaces, Myron A. Swayze, Director of Research, Lone Star Cement Corp., New York, N. Y., will present a paper emphasizing that proper finishing and curing are the key to durable surfaces.

Construction topics

Construction topics scheduled for the program include architectural concrete, lightweight monolithic concrete construction and an aggregate cooling plant for mass concrete.

J. A. Murlin, of George L. Dahl, Architects and Engineers, Dallas, Texas, will demonstrate the economies that were possible by using lightweight expanded shale concrete plus a flat-plate design in the construction of a two-story office building.

H. H. Roberts, Consolidated Builders, Inc., Mill City, Ore., will describe the aggregate and cement cooling plant used for mass concrete on Detroit Dam. To maintain proper temperatures in the 1,500,000 cu yd of concrete in Detroit Dam, the coarse aggregate was immersed in water 35 F until it cooled to 38 F, then drained and screened. For sand and cement there was no direct contact with the cooled water; they were cooled in continuous hollow-flight screw conveyors around which 35 F water was circulated.

Design and construction problems in architectural concrete will be presented by Prof. Michael Goodman, University of California, Berkeley, Calif.

Materials and properties

Premature stiffening of concrete and concrete durability will be discussed at the session on materials and properties of concrete. Other papers will cover ultrasonic testing of concrete structures and studies of inhibitors for alkali-aggregate expansion.

False set of cement causes difficulties in mixing and placing and even though the stiffening is eliminated by job conditions, extra mixing or the addition of corrective admixtures, undesirable effects on the hardened concrete remain. R. F. Blanks and J. L. Gilliland, Bureau of Reclamation, Denver, Colo., will present a paper on the effects of premature stiffening on concrete and suggest correctives in the manufacture of cement to alleviate false set.

E. A. Whitehurst's paper from Purdue University, Lafayette, Ind., will describe the use of the Soniscope to study the physical condition of concrete structures. Bridges, dams, pavements and a navigation lock were studied.

J. J. Shideler, H. W. Brewer and W. Chamberlin, Bureau of Reclamation, Denver, Colo., will report an investigation that indicates that the amount of winter protection can be reduced when air-entraining concrete is used. Another paper from the Bureau by Mr. Brewer and R. W. Burrows will discuss laboratory tests that indicate that coarse-ground cement produces more durable concrete than fine-ground cement.

West Coast committee

The local arrangements for the San Francisco meeting are under the direction of Harmer E. Davis, Director of the Institute of Transportation and Traffic Engineering, University of California, Berkeley, acting as general chairman.

The committee includes W. F. Arata, J. I. Ballard, Hugh Barnes, Raymond E. Davis, T. P. Dresser, E. A. Dufford, T. Jack Foster, Michael Goodman, John J. Gould, Homer M. Hadley, James Hirst.

Robert Horonjeff, Ed Howard, Walter L. Huber, J. E. Jellick, J. W. Kelly, W. A. Marsh, F. E. McCaslin, Malcolm McIntyre, William W. Mein, Jr., Robert L. Miller, Carl E. Monismith, Walter H. Price.

Lee J. Rothgery, S. W. Russell, H. W. Sayre, Wayne H. Snowden, Thomas E. Stanton, G. F. Steigerwalt, A. G. Streblow, Bailey Tremper, Byron Weintz, Harry A. Williams and Robert W. Winters.

See you in San Francisco!

Use This New-Billet Bar for Maximum Bond

To be sure of maximum bond strength in concrete, use the improved Bethlehem Reinforcing Bar.

The Bethlehem Bar provides maximum bond because of the tenacious gripping action of its specially designed lugs. These lugs are alternately sloped and uniformly spaced, and meet the deformation requirements of ASTM Specification A305. The firm anchorage they make possible means that wide tensile cracks in the concrete are unlikely to occur.

Bethlehem Reinforcing Bars are rolled as round sections, and are made from new-billet steel which meets ASTM Specification A15. In conformance with Simplified Practice Recommendation R26-50 of the United States Department of Commerce, the size of each bar is designated by a number, branded on each deformed bar to facilitate field identification. No change has been made in either the standard unit weights or nominal cross-sectional areas—therefore there is no need to alter basic design practices. Bar numbers, old size designations, areas and diameters are shown in the table below.

Our new reinforcing bar descriptive sheet gives concise information about the Bethlehem Bar. Keep one handy. Ask the nearest Bethlehem sales office for your copy. Or drop a line to us at Bethlehem, Pa.

BETHLEHEM STEEL

BETHLEHEM STEEL COMPANY, BETHLEHEM, PA.

On the Pacific Coast Bethlehem products are sold by Bethlehem Pacific Coast Steel Corporation. *Export Distributor:* Bethlehem Steel Export Corporation

NEW BAR NUMBER	2	3	4	5	6	7	8	9	10	11
OLD SIZE	1/4 in. ROUND NOT DEF.	3/8 in. ROUND	1/2 in. ROUND	5/8 in. ROUND	3/4 in. ROUND	7/8 in. ROUND	1 in. ROUND	1 in. SQUARE EQUIV.	1-1/8 in. SQUARE EQUIV.	1-1/4 in. SQUARE EQUIV.
Nominal Area (Sq In.)	.05	.11	.20	.31	.44	.60	.79	1.00	1.27	1.56
Nominal Round Diam.	.250	.375	.500	.625	.750	.875	1.000	1.128	1.270	1.410

BETHLEHEM REINFORCING BARS

The

KELLEY COMPACTOR POWER FLOAT

for

Surfacing, compacting, densifying and finishing concrete wearing surfaces

The weight of the machine, plus the action of the compacting hammers on the rotating disc COMPACTS and DENSIFIES lower sand and low-water-cement ratio mixes throughout the slab thickness.

DURABILITY INCREASED **SHRINKAGE REDUCED**

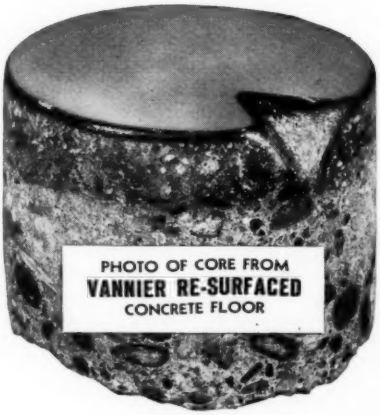
For Information write:

KELLEY ELECTRIC MACHINE CO.

287 Hinman Avenue **Buffalo 23, N. Y.**

FACTORY AND WAREHOUSE FLOORS
SUCCESSFULLY RE-SURFACED

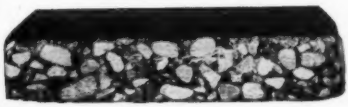
It is practical and economical, through the Vannier method, to re-surface structurally sound concrete floors.



← METALLIC WEARING SURFACE

← NEW BONDED CONCRETE FINISH

← OLD STRUCTURAL SLAB



Cross Section of VANNIER Asphaltic Concrete Wearing Surface showing Aggregate Arrangement Designed to Carry Maximum Loads—both Moving and Static.

The VANNIER METHOD
of Re-surfacing Concrete with Concrete

TYPICAL INSTALLATIONS

Fisher Body Division G-M
Flint, Mich.

St. Regis Paper Co.
Deferiet, N. Y.

American Bakeries Co.
Charlotte, N. C.

Thatcher Mfg. Co.
Streeter, Ill.

Trico Products Corp.
Buffalo, N. Y.

is practical, not only from the standpoint of DURABILITY but because of the important SAVING IN TIME. The training and experience of our mobile field units, cooperating with production personnel, enable us to resurface floors in operating plants without interrupting production schedules.

Send for descriptive, illustrated bulletins . . . or we will be glad to make a survey of your floor conditions and submit recommendations and quotations.

The VANNIER CO., Inc.
CONTRACTORS

DESIGN • **INDUSTRIAL FLOORS** • CONSTRUCTION

4430 BAILEY AVENUE BUFFALO 21, N. Y. Tel. AMherst 0828

Who's Who This Month

Walter H. Price

A general summary of "Factors Influencing Concrete Strength," p. 417, is by Walter H. Price, Head of Materials Laboratory, Bureau of Reclamation, Denver, Colo.

Long active in Institute affairs and technical committees, Mr. Price has been with the Bureau since 1930, associated with materials testing and research related to the construction of large concrete structures. He is chairman of ACI Committee 210, Resistance to Erosion in Hydraulic Structures, and Committee 613, Recommended Practice for the Design of Concrete Mixes.

His work with the Bureau has included hydraulic model studies in connection with spillway designs of Boulder, Madden and other dams, and two years on the structural design of Boulder Dam spillways and power plant. Since 1943 his work has been devoted entirely to materials testing and research.

E. A. Whitehurst

E. A. Whitehurst, Research Engineer, Joint Highway Research Project, Purdue University, Lafayette, Ind., is the author of "Soniscope Tests Concrete Structures," p. 433.

Presently employed as a research engineer and instructor, engaged in highway engineering research, Mr. Whitehurst was an associate research engineer with the Portland Cement Assn. during the work described in the paper. A graduate of Virginia Military Institute after three years in the Marine Corps, he joined PCA in 1947; in 1950 he joined the staff of the Joint Highway Research Project and entered the graduate school at Purdue. He received a master's degree in January, 1951. The highway research is a joint project of Purdue University Engineering Experiment Station and the State Highway Commission of Indiana.

H. F. Clemmer

H. F. Clemmer, author of "Effective Sealing of Concrete Pavement Joints," p. 445, has been an ACI Member since 1937 and is currently chairman of Committee 617, Specifications and Recommended Practice for Concrete Pavements and Bases.

Mr. Clemmer entered the highway engineering field early in his career and has been associated with design and research for many years. In his present position as Engineer of Materials for the District of Columbia, he has direct charge of inspection and testing of all construction materials used by the Engineer Department.

He received BS and CE degrees from Iowa State College and later served on the staff, progressing to associate professor of civil engineering. Mr. Clemmer then entered the highway field and was in charge of the research work on the widely known Bates Experimental Road, one of the earliest comprehensive studies of highway design.

An active member of many technical societies including ASTM, AASHO, ASCE, ARBA and Highway Research Board, Mr. Clemmer is also general chairman of the joint committee of the ARBA and Highway Research Board on highway equipment.

Joseph J. Shideler, Harold W. Brewer and Wilbur H. Chamberlin

Already familiar to JOURNAL readers, the three authors from the Bureau of Reclamation, Denver, Colo., Joseph J. Shideler, Harold W. Brewer and Wilbur H. Chamberlin report tests that indicate "Entrained Air Simplifies Winter Curing," p. 449.

The most recent contribution of Messrs. Shideler and Chamberlin appeared in the December 1949 JOURNAL and Mr. Brewer was co-author of a paper in the January 1951 JOURNAL.

Mr. Shideler joined the Bureau in 1939 and has been associated with investigations of thermal and physical properties of concrete for Bureau structures. Mr. Chamberlin also joined the Bureau staff in 1939 after serving with the Corps of Engineers, U. S. Army, for seven years. While with the Bureau, he has been mainly interested in the testing of concrete to determine length

Darex AEA

is a catalyzed air entraining agent, *specifically formulated* for making air entrained concrete.

Darex AEA

is being used in all types of concrete work, all over the world.
Distribution warehouse stocks are conveniently located throughout North and South America and in most foreign countries.

In the Mansfield Hollow Dam, Natchaug River, Connecticut, Darex AEA concrete provides better placeability, reduces bleeding and segregation . . . increases durability with its greater resistance to freezing and thawing.

DEWEY AND ALMY CHEMICAL COMPANY
CAMBRIDGE 40, MASSACHUSETTS
CHICAGO 38 MONTREAL 32

DAREX REG. U. S. PAT OFF

changes, flexural properties and dynamic modulus under various curing conditions.

Also a materials engineer, Mr. Brewer has been investigating concrete durability. He joined the Bureau in 1933 and has been in charge of freezing and thawing investigations for ten years.

Otto Safir

Mr. Safir describes recent experiences in "Precast Concrete Construction in Canada," p. 461. After graduation in engineering in 1931, Mr. Safir did postgraduate work in Switzerland on reinforced concrete bridges. Then, until 1945, he worked with consulting engineers and designing contractors in England and had his own consulting practice from 1945-49. His main interests have been structural development and he has been responsible for the design of several reinforced concrete bridges. In 1945 he designed one of the first precast structures in England, a double bay shed with two 75-ft spans. He was also associated with various precast concrete housing and factory projects in England. Since moving to Canada in 1949 he has engaged in similar work there, notably in British Columbia.

M. Fornerod

M. Fornerod discusses "Factors in Prestressed Girder Design," p. 469, based on experience with the Walnut Lane Bridge, Philadelphia. Mr. Fornerod's interest in bridge design goes back to his work from 1931-33 as design engineer in the office of Robert Maillart in Switzerland, whose outstanding bridge projects are well known.

A 1926 graduate of the Federal Polytechnic Institute of Zurich, Switzerland, Mr. Fornerod first spent two years in Spain as construction superintendent on a large reclamation project. Subsequently he designed reinforced concrete industrial buildings and foundations for White Construction Co., New York, and later joined the engineering staff of Electric Bond and Share Co. where he designed concrete structures for hydro-electric and steam power plants.

Following the association with Maillart, Mr. Fornerod entered consulting practice specializing in the design of reinforced concrete structures, one of which was a 300-ft spandrel bridge. During this period he was commissioned by the Swiss Foundation for Scientific Research to analyze the stresses and measure strains on Maillart's famous "Schwandbach" bridge, a thin-shell arch bridge of exceptional proportions.

After an extended trip through the Middle East in connection with important bridge and irrigation projects in Iraq, he became assistant to the general manager of the Bosari Tank Corp. of America and studied the design of reinforced concrete multistory pressure tanks.

Since 1943 Mr. Fornerod has been with the Preload Corp., which specializes in the design and construction of prestressed concrete structures. Chief engineer of the firm since 1946, he has been active in the development of modern design methods for prestressed thin circular and spherical shells for tank walls and dome roofs. Following an exploratory trip to Europe in 1947 he became active in the design of prestressed concrete bridges of which the Walnut Lane Bridge is the first major one to be built in this country.

Mr. Fornerod is a member of ACI, ASCE, NSPE and the International Assn. for Bridge and Structural Engineering.

• • •

Rising costs make it a necessity . . . .

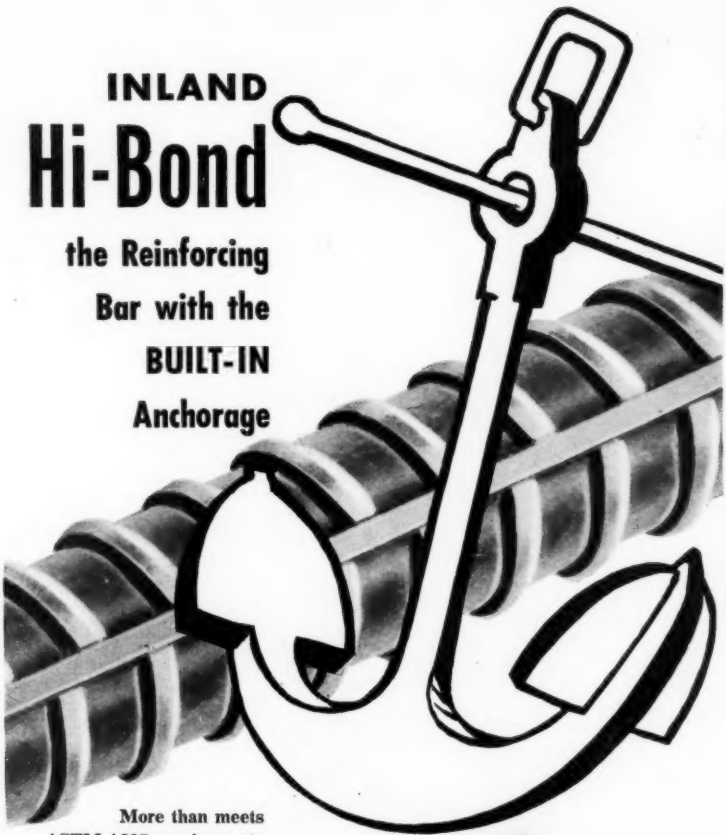
Contingent upon approval by the San Francisco convention of proposed increases in dues (See January News Letter for proposed revision to ACI By-Laws), the subscription price of the JOURNAL will be increased from $10.00 to $12.00 effective the first of the JOURNAL Volume year No. 23, September 1951.

• • •

INLAND

Hi-Bond

the Reinforcing Bar with the BUILT-IN Anchorage



More than meets ASTM A305 requirements

•

Stocked by
Joseph T. Ryerson & Son, Inc. Warehouses throughout the middle west and west.

•

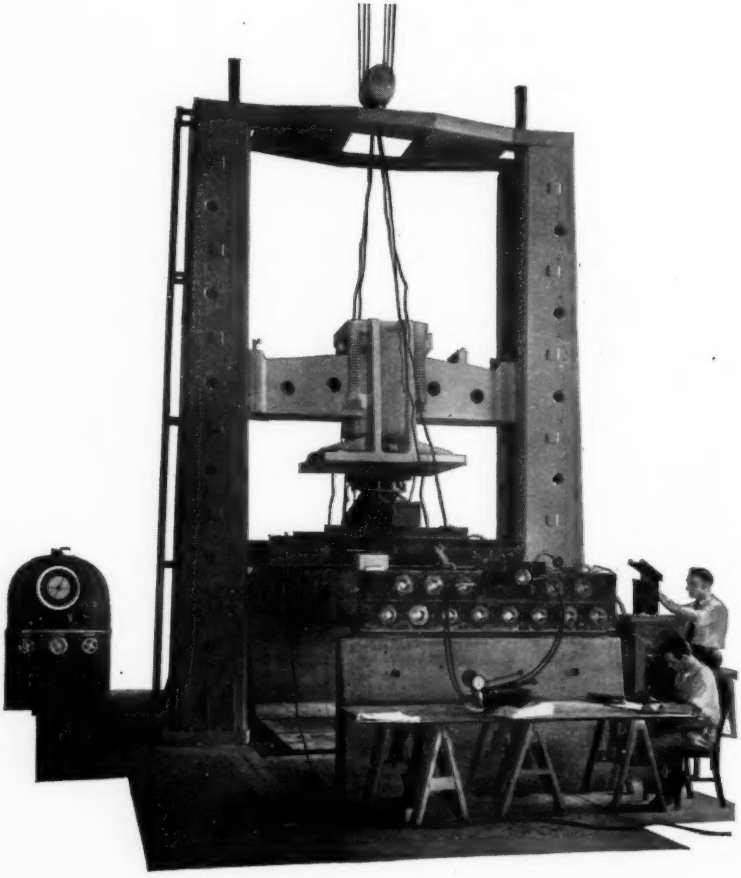
Other Products:
Bars • Structurals • Plates • Sheets Strip • Tin Plate • Floor Plate • Piling • Rails • Track Accessories

INLAND

INLAND STEEL COMPANY
38 S. Dearborn St., Chicago, Ill.

Sales Offices: Chicago, Davenport, Detroit, Indianapolis, Kansas City, Milwaukee, New York, St. Louis, St. Paul

"One of our most useful
The BALDWIN



pieces of equipment— 1,000,000# machine"

. . . that's what an official of the Portland Cement Association's Research and Development Laboratories says about this new Baldwin which permits actual service loads in flexure and compression to be applied to *full-scale* concrete structural members and assemblies.

Useful is right! With a capacity of 1,000,000 lbs. . . . a maximum clear space between columns of 10 feet and between bedplate and cross-head platen of 16 feet . . . a foundation design to accommodate structures or beams up to 30 feet between supports . . . this new Baldwin testing machine opens a complete new field of opportunity in improving the design of concrete members and structures.

Tests like this one illustrated have already demonstrated the benefits of the new machine. Here a 20-ton prestressed railroad trestle slab, 25 feet long, 7 feet wide and 1 foot 6 inches deep, is being loaded to failure, which occurred at 2.8 times design load.

The new possibilities are further extended by another Baldwin testing device. . . the SR-4® strain gage. Over 100 of these gages were distributed at selected points throughout the slab. They were connected to a central "switchboard" so that readings from each station could be taken at each increment of loading. By providing a complete picture of stress distribution in the slab, the gages furnish invaluable data to the designer.

You will find Baldwin Concrete Testing Machines helpful aids in plant,

highway and construction laboratories for routine and specification testing. For further information on the advantages of these Concrete Testing Machines you are invited to write for the following bulletins:

Bulletin 327—on the Baldwin 100,000 lb. Concrete Testing Machine.

Bulletin 318—on the Baldwin 300,000 lb. Concrete Testing Machine.

BALDWIN-LIMA-HAMILTON CORPORATION
Philadelphia 42, Pa., U. S. A.

Offices: Chicago, Cleveland, Houston, New York, Philadelphia, Pittsburgh, San Francisco, St. Louis, Washington

In Canada: Peacock Bros., Ltd., Montreal

BALDWIN BLH LIMA-HAMILTON

BALDWIN-LIMA-HAMILTON

TESTING HEADQUARTERS

Honor Roll

February 1, 1950 to December 31, 1950

Five percent of ACI's membership is represented on the Honor Roll for this year.

Those listed are to be commended. Similar member-getting activity by a large part of the remaining 95 percent would assure continuing healthy growth in membership, committee activity and publications.

Newlin D. Morgan (Ill.) **45½**
Newlin D. Morgan, Jr. (Wyo.) **22**
Leon Venegas (La.) **10**
Rafael Ruiz P. (Guatemala) **9**
Henry E. Griset (N. C.) **7**
E. I. Fiesenheiser (Ill.) **6½**
R. H. Sherlock (Mich.) **6½**
Chester P. Siess (Ill.) **6½**
George C. Alden (Calif.) **5½**
A. Amirikian (Md.) **5**
Charles G. Peller (Ind.) **5**
Luis A. Pietri-Lavie (Venezuela) **5**
Andrew Reti (Venezuela) **4½**
E. W. Bauman (D. C.) **4**
Earl I. Brown (Ga.) **4**
Phil M. Ferguson (Texas) **4**
Frank H. Jackson (D. C.) **4**
Henry L. Kennedy (Mass.) **4**
James A. McCarthy (Ind.) **4**
Oscar A. Nunez (Venezuela) **4**
Cesar Oliver-Rugeles (Venezuela) **4**
Byram W. Steele (D. C.) **4**
Ivan M. Viest (Ill.) **4**
James L. Atkinson (Calif.) **3½**
F. Thomas Collins (Calif.) **3½**
Castor Segundo Goa (Venezuela) **3½**
Samuel Hobbs (Calif.) **3½**
Wm. T. F. Hooper, Jr. (Ill.) **3½**
Melvin W. Jackson (Ga.) **3½**
S. Moore (Canada) **3½**
Clayton M. Crosier (Kans.) **3**
Raymond E. Davis (Calif.) **3**
Ashby T. Gibbons, Jr. (Ga.) **3**
Gerald K. Gillan (Mo.) **3**
Emil A. Gramstorff (Mass.) **3**
Walter N. Handy (Ill.) **3**
M. D. Hicklin (Ore.) **3**
J. E. Jellick (Calif.) **3**
G. Millsom (Canada) **3**
Aloysius E. Cooke (Conn.) **2½**
Rodolfo C. Garcia (Philippine Islands) **2½**
H. J. Gilkey (Ia.) **2½**
Orlando W. Irwin (Ill.) **2½**
W. H. Jacobs (Ill.) **2½**
S. J. Warberg (Mich.) **2½**
George D. Youngclaus (Calif.) **2½**
Ralph L. Barbehenn (N. J.) **2**
Roberto Barillas F. (Guatemala) **2**
Sterling Lowe Bugg (Fla.) **2**
W. Fisher Cassie (England) **2**
J. J. Creskoff (D. C.) **2**
Belmon U. Duval (Ohio) **2**
Eddy N. Hernandez (La.) **2**
J. Sterling Kinney (N. Y.) **2**
George E. Large (Ohio) **2**
William Lerch (Ill.) **2**
F. J. Ochoa U. (Guatemala) **2**
Leroy A. Staples (La.) **2**
J. Neils Thompson (Texas) **2**
A. G. Timms (D. C.) **2**
Hugh Barnes (Calif.) **1½**
J. F. Baxter (N. Y.) **1½**
Oscar Benedetti (Venezuela) **1½**
Anthony Henry Clark (England) **1½**
Raymond E. Davis, Jr. (Calif.) **1½**
Charles W. DeGroff (N. Y.) **1½**
Harlan H. Edwards (Wash.) **1½**
H. F. Gonnerman (Ill.) **1½**
Ernst Gruenwald (N. Y.) **1½**
George E. Hatch (Hawaii) **1½**
Elmo C. Higginson (Colo.) **1½**
Leo M. Legatski (Mich.) **1½**
H. J. McGillivray (Fla.) **1½**
J. Neil Mustard (Canada) **1½**
Henry A. Pfisterer (Conn.) **1½**
James E. Schumann (Calif.) **1½**
Thomas C. Shedd (Ill.) **1½**
Howard Simpson (Mass.) **1½**
John H. Thornton (Scotland) **1½**
G. K. Viall (Wis.) **1½**
Jose Antonio Vila (Cuba) **1½**
Charles E. Wuerpel (Ill.) **1½**
Roderick B. Young (Canada) **1½**
J. B. Alexander (Calif.) 1
F. O. Anderegg (N. J.) 1
Edward L. Ashton (Ia.) 1
Eugene D. Ayer (Colo.) 1
F. Barona (Mexico) 1
Hans H. Bleich (N. Y.) 1
Woodrow L. Burgess (Ore.) 1
E. F. J. Clark (Canada) 1
Roy R. Clark (Ore.) 1
Kenneth W. Clayton (Australia) 1
W. S. Cottingham (Wis.) 1
M. A. Craven (New Zealand) 1
R. A. Crysler (Canada) 1
Walter T. Daniels (D. C.) 1
Harmer E. Davis (Calif.) 1
DeWitt Dieterich (Ill.) 1
Joseph Di Stasio (N. Y.) 1
A. H. Douglas (Canada) 1
H. B. Emerson (Ill.) 1
C. C. Everhart (Ia.) 1
Stanley Gordon Fisher (Canada) 1
George W. Ford (Fla.) 1
Jose M. Franco-Cancio (Puerto Rico) 1
William E. Gabelman (Calif.) 1
Randall L. Gomien (Ohio) 1
Otto Graf (Germany) 1
Nathaniel Grant (Mo.) 1
Gilberto Guardia (Panama) 1
Horace P. Hamlin (N. Y.) 1

Proof...

Dorena Dam
U S Corps of Engineers
Dorena, Oregon
Proof ..that you use less Protex!

Walla Walla Dist. Corps of Eng. Photo
McNary Dam
U.S Corps of Engineers
Plymouth, Washington
Proof...that the larger projects specify Protex!

...that

Protex

AIR ENTRAINING AGENT

costs less to use!

Angostura Dam
U S. Bureau of Reclamation
Hot Springs, South Dakota
Proof...of Protex leadership in Air Entraining Agents!

More large projects specify Protex.

Ft. Gibson Dam
U.S Corps of Engineers
Muskogee, Oklahoma
Proof...that Protex gives consistent control!

Protex meets the requirements of Federal Specifications SS-C-192, ASTM Spec. C-175-48T.

AUTOLENE LUBRICANTS
COMPANY
INDUSTRIAL & RESEARCH DIVISION
1335 W. EVANS AVENUE
DENVER 9, COLO.

Autolene Lubricants Co.
1335 W. Evans Ave.
Denver 9, Colo.

Please send free booklet "The Facts on the Modern Placement of Concrete through Air-Entrainment."

Name.....

Address.....

City.....*State*.....

Robert J. Hansen (Mass.).....1
Terrell R. Harper (Texas).....1
W. L. Harrell (Texas).....1
Charles E. Hawke (Canada).....1
Harry H. Hawley (Ohio).....1
Philip W. Helsley (Calif.).....1
Bernard C. Herring (England).....1
Wallace E. Jobusch (Ind.).....1
William R. Johnson (Ore.).....1
Edgar R. Kendall (Nebr.).....1
Thomas B. Kennedy (Miss.).....1
Frank Kerekes (Ia.).....1
John C. King (Ohio).....1
K. V. Kini (India).....1
R. A. Kirkpatrick (N. Y.).....1
Blas Lamberti (Venezuela).....1
Demetrio Lausell (Puerto Rico).....1
David V. Lewin (Ohio).....1
Harry E. Lewis (N. Y.).....1
Alvin C. Loewer, Jr. (Pa.).....1
Winthrop E. Luke (Mass.).....1
Charles L. Luther (Ky.).....1
Ian Macallan (New Zealand).....1
Glenway Maxon (Wis.).....1
B. H. Maynard (Calif.).....1
Frank E. McClure (Calif.).....1
G. T. McCoy (Calif.).....1
J. H. Murdough (Texas).....1
J. A. Murlin (Texas).....1
James A. Murray (Mass.).....1
Amado B. Navoa (Philippine Islands) ...1
George A. Norwood (Texas).....1
William D. Nowlin (Va.).....1
Carlos Ospina (Colombia).....1
Wm. Otcovsky (Ill.).....1
Kenneth K. Paget (Canada).....1
Jose Antonio Pardo B. (Mexico).....1
Dean Peabody, Jr. (Mass.).....1
Howard F. Peckworth (Ill.).....1
Stanley A. Phillip (Ill.).....1
Harry W. Piper (Md.).....1
Dan H. Pletta (Va.).....1
Herman G. Protze (Mass.).....1
Frederick A. Reickert (Ill.).....1
Theodore O. Reyhner (Colo.).....1
Paul F. Rice (Mich.).....1
Evan L. Richard (Australia).....1
Frank E. Richart (Ill.).....1
Horace B. Rickey, Jr. (La.).....1
Bernard L. Robinson (N. Y.).....1
Gilbert H. Rowe (S. C.).....1
Quentin Rust (Calif.).....1
Otto Sauermann (Ill.).....1
Emil Schmid (N. J.).....1
Wilfrid Schnarr (Canada).....1
Elwyn E. Seelye (N. Y.).....1
E. R. Smallhorn (Canada).....1
Howard R. Staley (Mass.).....1
Kenneth G. Stevens (S. Rhodesia).....1
M. Eugene Sundt (N. M.).....1
Pascal L. Tasio (N. Y.).....1
Wm. H. Thoman (Colo.).....1
H. F. Thomson (Ill.).....1
E. W. Thorson (Colo.).....1
LeRoy A. Thorssen (Canada).....1
G. E. Troxell (Calif.).....1
Gregory P. Tschebatarioff (N. J.).....1
Maurice P. Van Buren (N. Y.) 1
F. E. Votaw (Mass.).....1
A. G. Watt (Pa.).....1
W. R. Waugh (D. C.).....1
C. A. Willson (N. Y.).....1
Merle D. Wilson (Ohio).....1
Arthur G. Wood (Colo.).....1
S. H. Woodard (N. Y.).....1
A. E. Wynn (S. Africa).....1
Aubrey D. Young (La.).....1
Edward F. Young (Minn.).....1
R. A. Zern (Pa.).....1

The following credits are, in each instance, "50-50" with another member.

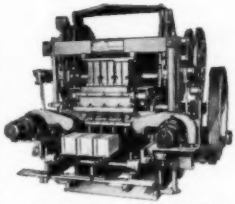
James Adam, Jr.
Herbert C. Allen
Frederick G. Anderson
R. Howard Annin
Charles D. Babcock
Malcolm F. Baker
C. Merrill Barber
Joseph L. Bargar
George E. Barnes
Frank Baron
A. Allan Bates
K. P. Billner
G. L. Blanchard
A. E. Bond
Wallace G. Bond
Harry E. Born
J. G. Bragg
James E. Branch
Charles Britzius
Carlos D. Bullock
S. D. Burks
R. A. Burmeister
Harold J. Buttery
D. J. Cameron
Neal J. Campbell
Julian B. Carson
Harold Carter
S. J. Chamberlin
Jules P. Channing
Ralph E. Coblentz
Miles K. Cooper
Roger H. Corbetta
William A. Crabb
A. E. Cummings
Clayton L. Davis
Robert G. Deitrich
G. A. DeLong
Harry Delzell
R. G. Douglas, Jr.
Albert W. Dudley
Leonard E. Dunlap
John R. Dwyer
Harry Ellsberg
W. J. Emmons
L. G. Farrant
Jorge Figuls
John D. Freeman
P. J. Freeman
Bengt Friberg
Bernardo Fuentes A.
Mario W. Gamalero
J. H. Gandhi
E. L. Gardner
A. H. Garnsey
Grayson Gill
A. T. Goldbeck
O. H. Gosswein
Alex H. Graves
Homer M. Hadley
Albert Haertlein
Theodore F. Hagerman
William H. Hall
W. C. Hansen
Harvey L. Harris
Roy Hawkins
John M. Hayes
Eivind Hognestad
W. A. Hohlweg
William N. Holway
Wilfred M. Honour
E. S. Horne
J. W. Hubler
Virgil L. Hughes
Maurice Hunt
Anthony Hunter
S. E. Hyde, Jr.
Raja A. Iliya
W. R. Ingram
C. E. Ireland
C. L. N. Iyengar
Arthur M. James
M. E. James
Bruce M. Johnson
J. G. Johnson
H. D. Jolley
Bernerd D. Keatts
J. W. Kelly
Clyde Kesler
W. D. Kimmel
W. P. Kinneman
Ralph W. Kluge
Theodore C. Knight
Jean H. Knox
M. C. Kolinski
Paul P. Kraai
Richard J. Kroc
W. G. Limbach
J. E. Lothers
Irvin H. Luke
I. D. MacKenzie
M. F. Macnaughton
James V. Mandia
Eastman M. Markell
W. T. McClenahan
Sam McCluer
C. T. J. McDowell
A. J. McElrath
Douglas McHenry
Harmon S. Meissner
Willis T. Moran
E. J. Mueller
Glenn Murphy
V. S. Murray
A. E. Myers
William T. Neelands
Henry L. Neve
Walter Newland
Roy A. Nyquist
Raymond G. Osborne
John P. Ottesen
Douglas E. Parsons
Orley O. Phillips
Thomas Poli
Milos Polivka
James A. Polychrone
Chesley J. Posey
Leo F. Pratt
Walter H. Price
Charles J. Prokop
Raymond C. Reese
R. L. Reid
Ross M. Riegel
Guy Rinfret
Charles S. Rippon
Richard A. Roberts
A. Rodriguez-Delfino
Pedro Rodriguez
Paul Rogers
A. W. Rohlwing
Mark R. Rosumny
Oswaldo Rovati B.
William H. Rowan

Continued on p. 32

BESSER VIBRAPAC BLOCK

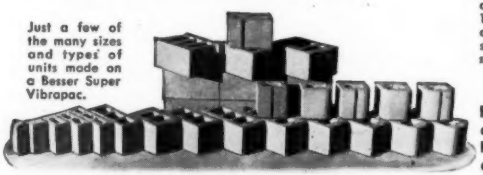
FOR ALL TYPES OF STRUCTURES

To the BEAUTY and PERMANENCE of Besser Vibrapac Block must be added the VERSATILITY of this modern building material. Its many attractive and distinctive designs, as well as variety of sizes, can be applied equally well to any type of structure. Vibrapac control of block density and texture permit the manufacture of units required for insulation, soundproofing and acoustical control. The durability and design of Besser-made concrete masonry units stand as a building beacon to architects and builders throughout the world.



BESSER SUPER VIBRAPAC — the fully automatic, Plain Pallet block machine. Requires no machine operator. Pallets are fed automatically. Three 8" x 8" x 16" block are made at a time on one Plain Pallet. Smaller sizes in larger multiples on the same pallets.

Just a few of the many sizes and types of units made on a Besser Super Vibrapac.



Only ONE set of Plain Pallets required for ALL types and sizes of block. Speeds up block production. Reduces costs.

BESSER Vibrapac Better Concrete Masonry Units, in the Hands of Skilled Architects and Builders, Have Become in Effect a New Building Material.

BESSER MANUFACTURING CO.

Complete Equipment for Concrete Products Plants

135 - 50th STREET · ALPENA, MICHIGAN, U.S.A.

A 5539-1P

New Members

California was top membership state for the month of December providing six new Member applicants. New York was second with four applicants.

New Member applicants for December—37—total membership Jan. 1, 1951, 5215.

California

Alameda, City of, (Corp.) Alameda, Calif.
French, Stanley J., (Jr.) 4369 Witherby, San Diego 3, Calif.
Graves, Michael, (Indiv.) c/o R. Howard Annin, Rm. 803, 408 S. Spring St., Los Angeles 13, Calif.
Irwin, L. W., (Indiv.) 7909 E. 4th St., Downey, Calif.
Jennings, George H., (Indiv.) 2131 University Ave., Berkeley 4, Calif.
Lorman, William R., (Indiv.) Box 669, R. R. No. 2, Oxnard, Calif.

Colorado

Whitaker, E. C., (Indiv.) 1672 S. Adams St., Denver, Colo.
Ziegler, Melvin H., (Indiv.) Adolph Coors Co., Golden, Colo.

Connecticut

Silliman, Frederick B., (Indiv.) 290 North Ave., Bridgeport, Conn.

Florida

Edwards, Harry H., (Indiv.) Lakeland Engineering Associates, Inc., Box 495, Lakeland, Fla.

Georgia

Ganchar, Sol, (Indiv.) 416 E. 57th St., Savannah, Ga.

Illinois

Adkins, Lonnie O., (St.) 306 E. Hill St., Champaign, Ill.

Iowa

Garton, C. E., (Indiv.) 120½ Welch Ave., Ames, Ia.

Maryland

Penniman & Browne, Inc., (Corp.) 341 St. Paul Place, Baltimore 2, Md. (Donald W. Smith)

Minnesota

Western Mineral Products Co., (Corp.) 1720 Madison St., N. E., Minneapolis 13, Minn. (M. A. Johnston)

Missouri

Rushing, James Forrest, (Indiv.) 4522 Holly Ave., St. Louis 15, Mo.

New Jersey

Causing, Humbert, (St.) Room 314, 1901 Hall, Princeton University, Princeton, N. J.

New York

Ateshoglou, A. D., (Indiv.) Hotel St. George, Clark & Henry St., Brooklyn 2, N. Y.
Bergier, Frederic M., (Indiv.) 21-92 Palmer Ave., New Rochelle, N. Y.
Berman, Frank R., (Indiv.) 198 42 Pompeii Ave., Hollis 7, N. Y.
Pannone, Frank E., (St.) 1178—71st St., Brooklyn 28, N. Y.

Texas

Best, Gene, (Indiv.) 2300 Ave. Q, Snyder, Texas
Kettl, John W., (Indiv.) 1233 DuBarry Lane, Houston 18, Texas

Virginia

McDonough, James A., (Indiv.) Civil Engr. Dept., Virginia Military Institute, Lexington, Va.
O'Geary, Dennis Traylor, (Jr.) 915 Hillside Ave., Norfolk, Va.

Canada

Dean, M. F., (Indiv.) c/o Starr Manufacturing Works, Ltd., Dartmouth, N. S., Canada

England

British Cast Concrete Federation (Corp.) 17, Amherst Rd., Ealing W. 13, London, England

Guam

Bowen, James R., (Indiv.) Station No. 14, Guam, Guam

India

Rao, N. S. Govinda, (Indiv.) Prof. of Hydraulics & Civil Engr., Indian Institute of Science, Bangalore 3, India
Varghese, P. C., (Indiv.) Hirakud Research Station, Hirakud Dam Project, Hirakud, via Sambalpur, Orissa, India

Iraq

Hishmat, Abdul Kadir, (Indiv.) 5/8 Waziriyah, Baghdad, Iraq

Japan

Ono, Takenosuke, (Indiv.) 1st Engr. Faculty, Nihon University, Surugadai, Chiyoka-Ku, Tokyo-To, Japan

Continued on p. 32

7 types of CP VIBRATORS

for pneumatic or high-frequency electric operation, handle all kinds of placements of mass or reinforced concrete.

One-man pneumatic and high frequency electric, small diameter models for reinforced concrete, walls and columns, etc.

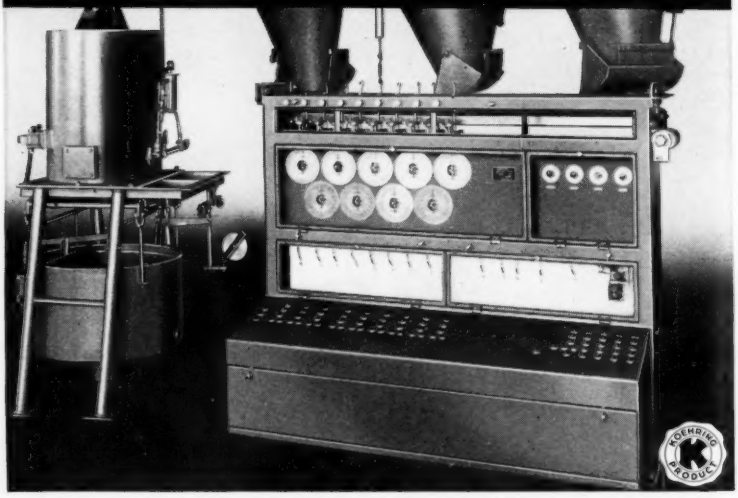
One-man and two-man pneumatic and high frequency vibrators for mass concrete.

Write for full information on complete line of CP Vibrators

CHICAGO PNEUMATIC TOOL COMPANY
General Offices: 8 East 44th Street, New York 17, N. Y.

PNEUMATIC TOOLS • AIR COMPRESSORS • ELECTRIC TOOLS • DIESEL ENGINES
ROCK DRILLS • HYDRAULIC TOOLS • VACUUM PUMPS • AVIATION ACCESSORIES

NERVE CENTER for RECORD PRODUCTION



JOHNSON Automatic Mixing Plants Offer YOU...

HIGH YARDAGE OUTPUT because . . .

Concentric zoning groups 1 to 6 mixers around center for most direct material flow, eliminates segregation of aggregates, and allows steep bottom slopes in aggregate hoppers for fast charging time. Central location of Cement Compartment permits steep slopes in cement tank for fast flow and maximum premixing action by introduction of cement into center of aggregate stream. Automatic Batcher Control features 2-step filling for fast and accurate weighing, instantaneous mix selection and fast automatic moisture compensation.

QUALITY CONTROL because . . .

Separate single-material Batcher units operated from a master control board (see photo above) are used for cement, water and each size aggregate. 2-step fill insures accurate weighing because intermittent feeding at close of charging operation reduces possibility of overload. Moisture compensator automatically corrects Water Batch scale to compensate for water content of sand. Adjustable retard on each Batcher gives optional delay in introducing Batch into mix.

ACCURATE PRODUCTION RECORDS because . . .

Chart pens operate directly from scale mechanism, record weight of each material on same sheet. Mixer consistency can also be shown. Electric counter records Batches discharged. Recorder also prints bars on chart paper, saving money and assuring accuracy of register.

THE C. S. JOHNSON COMPANY

Koehring Subsidiary Champaign, Illinois

Positions and Projects — ACI Members

Butler elected to Minnesota Senate

Gordon H. Butler, president of the Polaris Concrete Products Co., Duluth, Minn., was elected to the Minnesota State Senate in the November elections. Called to active duty as major in the Corps of Engineers in 1941, Mr. Butler was in charge of the construction of a $22,000,000 anhydrous ammonia plant in this country, and later served abroad in both theaters of war. A long-time ACI member, he is a past-president of the Duluth Engineers Club and the Minnesota Federated Engineering Societies, and a Director of ASCE.

Drexel alumni honor Friel

Francis S. Friel, president of the Philadelphia consulting firm of Albright and Friel, was given the fourth annual Drexel Alumni Assn. Award recently. The award is given for efforts on behalf of Drexel Institute or the alumni association. Mr. Friel is a member of Drexel's board of trustees.

Bleich named associate professor

Hans H. Bleich, lecturer in civil engineering at Columbia University since 1946, has been named associate professor. Professor Bleich is engaged in the completion of a research project on the buckling strength of metal structures, sponsored by the U. S. Navy in cooperation with the Column Research Council. For the past five years he has been associated with Hardesty and Hanover, New York City consultants.

Plummer elected vice-president

Fred L. Plummer, Director of Engineering, Hammond Iron Works, Warren, Pa., has been elected second vice-president of the American Welding Society for 1950-1951.

Robert Moorman

Robert Moorman, professor of civil engineering, University of Missouri, has been elected councillor from the central district of Chi Epsilon, honorary engineering fraternity.

Savage appointed to Point Four board

John L. Savage, for many years chief design engineer for the Bureau of Reclamation was appointed by President Truman to the twelve-member board for the Point Four Program. The board, which is headed by Nelson A. Rockefeller, will launch a program for "development of backward areas as an important means of working for ultimate peace and security in the world." Mr. Savage was instrumental in the design of Hoover and Grand Coulee dams and has served as consulting engineer to several foreign governments.

Karrer transferred

Wilfred L. Karrer, Bureau of Reclamation construction engineer on the Lewiston Orchards project in Idaho since 1947, has been transferred to a similar position on the Yakima project in eastern Washington. His major immediate job will be to supervise advance planning activities on the 21,000-acre Kennewick division of the Yakima project and also to supervise rehabilitation work on Tieton, Kachess, Bumping and Keechelus dams. Karrer joined the Bureau as a transitman on the Owyhee project in Oregon in 1929 after graduation in engineering from the University of California. Nine years later he transferred to the Chief Engineer's office in Denver as associate engineer. He became engineer on Anderson Ranch Dam of the Boise project in 1941, served in the Navy during the war and returned to the bureau as regional engineer in the Pacific Northwest.

Washington engineers elect officers

Stanton Walker, Director of Engineering, National Sand and Gravel Assn. and National Ready Mixed Concrete Assn., Washington, D. C., has been elected president of the Engineers Club of Washington.

This year's officers include several ACI Members besides Mr. Walker. Bernard F. Locraft is vice-president of the group and Yvone K. Howat is treasurer. The Board of Directors has three ACI Members represented, E. W. Bauman, Harold F. Clemmer and P. Y. K. Howat.

JACKSON VIBRATORS

For every purpose to which Vibrators are applicable in the concrete construction industry, we are confident we can supply equipment that will give you not only the best and fastest placement, but also the maximum of dependability and trouble-free service. Pioneers and outstanding developers of vibratory equipment, our complete line includes internal and external vibrators for: General Construction * Light Construction * Mass Concrete Dam Construction * Hard-to-get-at Places * Form Vibrating * Floors, Streets and Highways * Pipe Manufacturing * Movement of Materials—Vibratory Tables—Thorough soil compaction of bridge approaches, similar fills and concrete floor sub-bases, etc. Drop us a line for complete information.

JACKSON SIDE FORM VIBRATOR

Eliminates manual vibrating of concrete at side forms. Saves the better part of two men's labor. Mounts on any modern finisher or spreader. Employs two or more vibratory units that are simultaneously lowered into or raised from the concrete by the finisher operator. Units operate close to forms or reinforcement without fouling—ride over any obstruction encountered. Will not penetrate into sub-base. Assure thorough compaction regardless of speed of finisher or spreader. No spots are missed. Vibratory motors are integral with vibratory unit, transmitting vibration directly to loops of steel, thus eliminating all flexible shafting. Rugged, reliable and long lasting.

JACKSON ELECTRIC CONCRETE VIBRATOR

The Jackson Model FS-150A Electric, flexible shaft vibrator is an extremely advantageous vibrator for all general concrete construction. Its 2½ HP universal motor provides plenty of power to handle the stiffest mixes with the maximum length of shaft (28′). It may be operated from any light-socket, fused to 30 amps, supplying 115 volt, single phase AC or DC, or Jackson Portable Power Plant. 8,000 to 10,000 VPM. Choice of 3 vibrator heads. Easily converted to wet or dry grinding. Convenient, easy to handle.

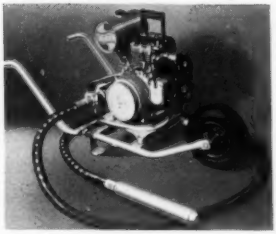
JACKSON PORTABLE POWER PLANTS

The most rugged, reliable plants in existence. Powered by nationally renowned engines, equipped with permanent-magnet generators which require no maintenance or adjustment, they produce both single phase and 3-phase 115 volt, 60 cycle AC. Ideal for operating all Jackson vibrators, contractor's power tools and lights. Three models: rated 1.25, 2.5 and 5 KVA (at unity power factor) in single phase, and 1.5, 2.5 and 7.5 KVA (at unity power factor) in 3-phase. The largest model is trailer-mounted for easy mobility. Skid mounting of all models is optional. Model M-2 (2.5 KVA) illustrated.

ELECTRIC TAMPER & EQUIPMENT CO.,

The Line that Solves EVERY Concrete Vibrator Problem to BEST Advantage

JACKSON FLEXIBLE SHAFT VIBRATOR



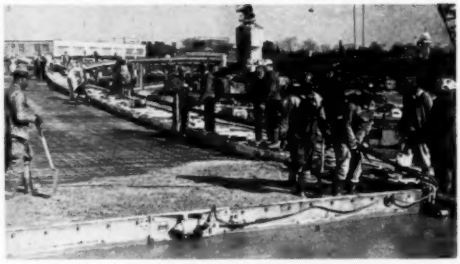
Compact and powerful (4 HP engine of national renown) built to take severe usage. Available with vibrator heads in 3 sizes and flexible shafting in 7′ and 14′ lengths up to 28′. Full-swivel, dirt-proof base. Furnished with or without wheelbarrow mounting. Frequency is instantly adjustable over a wide range. May be quickly converted to wet or dry grinding and drilling. An excellent vibrator for general construction work. Model FS-6A.

JACKSON VIBRATORY COMPACTOR

Nothing equals this machine for fast and thorough compaction of granular soils in those areas adjacent to: BRIDGES — CULVERTS — STRUCTURES — ABUTMENTS — UNDER FACTORY FLOORS — SIDEWALKS — RAMPS — MUNICIPAL PAVEMENTS — EARTH-FILL DAMS — TRENCHES — AND MANY OTHER PLACES. It is far faster and better than hand tamping and can be used in many places inaccessible to rollers. Achieves up to 95% of maximum density in a SINGLE PASS. Propels itself under normal conditions. Firmly compacts 15 to 18 sq. ft. per minute to a depth of 12″. Easily portable. Time-tested vibratory motor—3 phase, 110 volt, 60 cycle AC. Two of these machines may be operated simultaneously by one Jackson (1.25 KVA) Power Plant.

JACKSON VIBRATORY SCREED

Unequalled for municipal paving, highway widening and patching, bridge decks; etc. Will undercut at side forms, roll back for second passes, strike off crowns, both regular and inverted, permit operators to work from front, rear or sides. Has such strong tendency to propel itself that only small effort is necessary to strike off stiff mixes. Works right up to and around sewers, manholes and other obstructions. Does an excellent job of vibrating concrete to depth of 10 inches on any slab from 6 feet to any practical width Full slabs up tc 30 feet may be poured without center joints. Powered by Jackson 1.25 KVA Power Plant.



LUDINGTON, MICHIGAN

ACI Members to speak at British building conference

The program of the Building Research Congress, to be held in London, England, Sept. 11-20, 1951, will feature papers by eight ACI Members.

G. Wastlund, Cement and Concrete Institute, Stockholm, Sweden, will present a general review of the influence of modern research on structural design in introducing that session of the conference. In the same session A. R. Collins, Cement and Concrete Assn., London, England, will discuss research on, and the development of, prestressed concrete. F. Campus, Institut du Genie Civil, Liege, Belgium, and George Winters, Cornell University, Ithaca, N. Y., will speak on the design of welded steel structures and the influence of research on steel structural design.

In the section of the congress devoted to building materials Douglas E. Parsons, National Bureau of Standards, Washington, D. C., will discuss research on weathering and durability. Robert F. Blanks, Bureau of Reclamation, Denver, Colo., will speak on concrete quality control and H. F. Gonnerman, Portland Cement Assn., Chicago, Ill., is scheduled to discuss the durability of concrete structures.

Representatives from the staff of Albert Kahn Associated, Architects and Engineers, Detroit, Mich., will take part in a symposium on the different methods of factory construction and their costs in relation to functional factors.

The conference, sponsored by British professional institutions, technical societies, governmental departments and representative industrial federations, will be divided into three divisions. The sessions will include the engineering and structural aspects of building, building materials and the factors affecting the comfort and efficiency of the people using the buildings.

NSGA and NRMCA meet in New Orleans

The joint annual convention of the National Sand and Gravel Assn., and the National Ready Mixed Concrete Assn. was held in New Orleans, La., Feb. 12-15.

Discussions on financial practices, labor relations, emergency regulations and merchandising occupied a major portion of the program. A "Clinic in Calculating Concrete Proportions" was featured as well as a panel discussion on operating problems in the ready-mix industry. Researches in concrete aggregates were also described.

Frank A. Randall

Frank A. Randall, structural engineer of Wilmette, Ill., died December 2 at Evanston, Ill. A structural and consulting engineer in the Chicago area for 45 years, Mr. Randall was chairman of a committee which framed the structural sections of the new Chicago building code. He was also chairman of a local Joint Committee on Soil Mechanics and Foundations and at the time of his death headed the Underground Construction and Dispersion Sub-Committee of the Chicago Civil Defense Corps.

The author of a number of technical books and papers, Mr. Randall was a member of ACI, ASCE, ASTM, Western Society of Engineers, The Society of Naval Architects and Marine Engineers and of several fraternal, business and social organizations.

Hendrik Cornelissen

Hendrik Cornelissen, for a number of years design and construction engineer in the engineering firm, Cornelissen and Salzedo, at Barranquilla, Colombia, died recently. He was a graduate of Rensselaer Polytechnic Institute.

Ernest O. Sweetser

Ernest O. Sweetser, professor emeritus of the Washington University Civil Engineering Department, St. Louis, died January 18 at his home in University City, Mo.

Dr. Sweetser, who retired in 1947 as head of the Civil Engineering Department, joined the Washington staff in 1905 as an instructor. He was president of the St. Louis Section of ASCE in 1928 and was president in 1935 of the Engineers Club of St. Louis. He had been a member of ACI since 1925. He specialized in the study of bridge structures and building materials. Dr. Sweetser was graduated from the University of Maine in 1905, and he received an advanced degree of civil engineer there in 1912.

Presenting a *new standard* in AIR METERS!

The Techkote White Air Meter...

An outstanding new concept of functional efficiency in meters for measuring entrained air in concrete. It is unexcelled in precision construction and offers the following exclusive features:

1. Inspectors Test Gauge—Maximum accuracy
2. Gauge fully protected in aircraft-type shock-proof mounting
3. The Field Check Test Device is an integral part of Meter
4. Lightest in weight—only 12 lbs.
5. 2-piece Clamping Device; Simple—Fast—Positive
6. Damage to pot rim will not affect positive seal
7. Only 9 to 11 strokes required to produce initial pressure
8. No hazard of premature release of air into concrete chamber
9. Only 4 ounces of water required to operate
10. Streamlined Design — Easiest to Handle—Fastest to Operate—Easiest to Clean

TO BE SURE...USE THE FINEST!

TECHKOTE COMPANY
INCORPORATED
820 West Manchester Avenue · Inglewood, California

PROFESSIONAL CARD

L. COFF, Consulting Engineer
198 Broadway, New York 7, N. Y.
PRESTRESSED CONCRETE STRUCTURES
Design, Estimates,
Construction Methods, Supervision

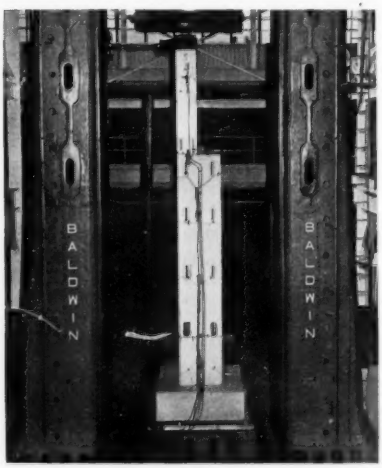
Pittsburgh Testing Laboratory names officers

A. R. Ellis, formerly president of Pittsburgh Testing Laboratory, Pittsburgh, Pa., has been elected chairman of the board of the company. R. C. Emmett, also an ACI Member and district manager for the firm, was named vice-president; C. M. Houck succeeded Mr. Ellis as president.

The large and the small of it

The largest and the smallest test loading and load measuring equipment known were used together recently by the U. S. Bureau of Reclamation to determine the load-carrying capacity of a reinforced concrete step-column 22 ft high, its weakest point under load, and the stresses and strains at numerous points on the surface and within the column.

With the load being applied to the pilaster and no load applied to the step, it took little more than one-third of the 5,000,000-lb loading capacity of the giant 3-story testing machine to cause failure. This took place immediately above the step under a load of 1,675,000 lb. Failure occurred through buckling of the longitudinal reinforcing steel, accompanied by shear and compression failure of the concrete.



Testing 22-ft concrete step-column

During application of the increasing load, SR-4 resistance wire strain gages, bonded to the concrete surface of the column and internally on reinforcing bars, measured the strains occurring at many points on the column by the change of electrical resistance in the wire grid. By means of a nearby Wheatstone bridge and amplifier circuit these changes were translated into inches of deformation per inch of length.

This test is typical of many to be undertaken by the Bureau on concrete structures, both full size and models. The column illustrated was designed for use in Bureau power plants.

Honor Roll

Continued from p. 22

John A. Ruhling
Frederic Rusche
Nicholas R. Samaha
Robert L. Sanks
R. W. Sauer
D. M. Schmid
Axel Schulze
James M. Shivley
J. Morgan Smith
Myron M. Smith
Ernest L. Spencer
R. A. Spencer
R. W. Spencer
John J. Stedje
John D. Stites
Herbert M. Stoll
Hale Sutherland
Ernest O. Sweetser
Thomas F. Taylor
L. W. Teller
Walter F. Tews
George P. Thigpen
J. Trueman Thompson
B. K. Thornley, Jr.
T. Thorvaldson
Paul M. Trueblood
Lee Turzillo
Lewis H. Tuthill
I. L. Tyler
Miguel Villa
E. H. Walker
Julius J. Warner
G. E. Warren
Willard W. Warzyn
L. T. Willoughby
Clement T. Wiskocil
Francis P. Witmer
A. E. Wood
Bourdette R. Wood
H. D. Worthington

New Members

Continued from p. 24

Philippine Islands

Buntian, Francisco V., (Indiv.) Philippine Ready Mix Concrete Co., Inc., P.O. Box 761, Manila, P. I.

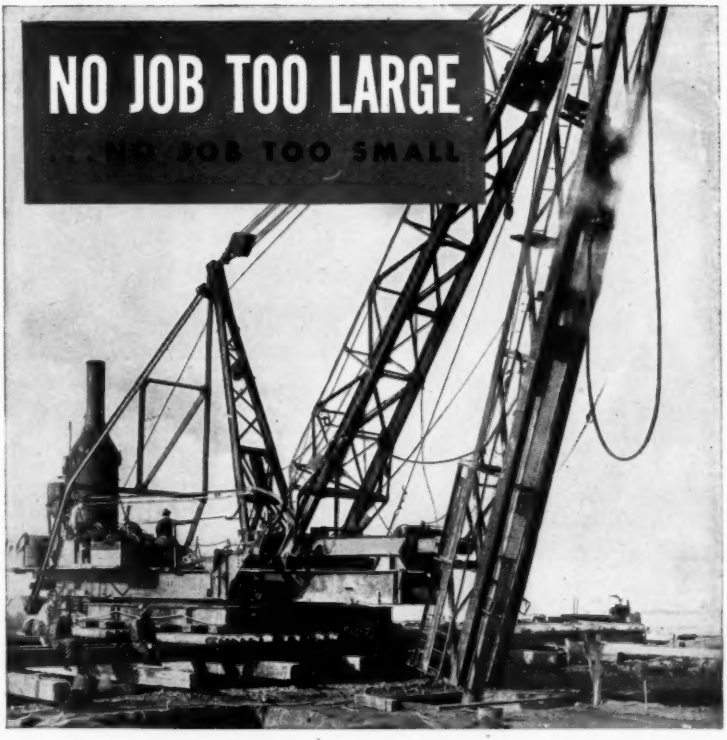
Del Rosario, Leopoldo C., (Indiv.) 2030 Yacal, Sta. Cruz, Manila, P. I.

Espiritu, Carlos M., (Indiv.) 164 Interior-1 Lipa St., Sampaloc, Manila, P. I.

Venezuela

Azcarate, Patricio de, (Indiv.) 2a. Avenida Bello Monte, Edificio Maracaibo, Apartamento 11, Caracas, Venezuela, S. A.

Lagrange Troconis, Gaston, (St.) Avenida Olimpo—Quinta La Casita, Urbanizacion San Antonio Caracas, Venezuela, S. A.



Owners, architects, engineers and general contractors know that buildings rest safely on Raymond concrete pile foundations. That's why for more than half a century on projects around the world Raymond concrete piles have been used again and again—to provide safe subsurface support that is dependably permanent. Your inquiries receive prompt attention.

SCOPE OF RAYMOND'S ACTIVITIES includes, in addition to borings for soil investigations, every recognized type of foundation construction—concrete, composite, precast, steel, pipe and wood piles. Also caissons; underpinning; waterfront construction and harbor and river improvements; shipbuilding facilities; and cement-mortar lining of water, oil and gas pipelines, 4" to 144" diameters, by Centriline Corporation, a Raymond subsidiary.

RAYMOND CONCRETE PILE CO.

Branch Offices in Principal Cities of United States and Latin America

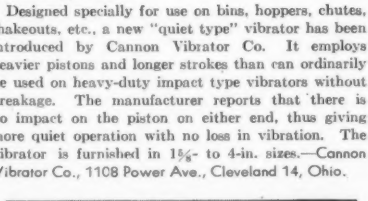
[illegible] STREET, NEW YORK 6, N. Y.

Tools, Materials, Services

Under this heading note will be made from time to time of producer literature of presumed technical interest (and available from its source for the asking) to ACI users of tools, equipment, materials, accessories and special services.

Bin and hopper vibrator

Designed specially for use on bins, hoppers, chutes, shakeouts, etc., a new "quiet type" vibrator has been introduced by Cannon Vibrator Co. It employs heavier pistons and longer strokes than can ordinarily be used on heavy-duty impact type vibrators without breakage. The manufacturer reports that there is no impact on the piston on either end, thus giving more quiet operation with no loss in vibration. The vibrator is furnished in 1⅝- to 4-in. sizes.—Cannon Vibrator Co., 1108 Power Ave., Cleveland 14, Ohio.



Powered wheelbarrow

New features added to the Prime-Mover, Model 15, wheelbarrow include forward direct drive with half speed reverse under power, constant mesh transmission and an increased load capacity of 1500 lb.

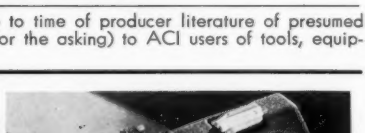
Two sizes of platforms with stakes are quickly interchangeable with the bucket. The manufacturer claims that ample power is available for taking steep ramps and working under continuous heavy duty. The unit is 31½ in. wide to permit access through doorways and has a turning radius of 33 in.—Prime-Mover Co., Muscatine, Ia.

Water repellent

A transparent water repellent, Monsoon, for brick, mortar and other masonry structures has been developed by State Chemical Corp. Containing a silicone resin, the manufacturer claims that the repellent can be applied indoors or outdoors over a wide temperature range with no change in appearance or porosity of the masonry. It is reported that the compound is self-cleansing and is effective against efflorescence.—State Chemical Corp., 1265 Broadway, New York 1, N. Y.

Bag packer

A unique feature of the Auger-Matic valve packer is the multiple auger which can be changed to fit the type of product being packed. The packer fills valve bags of 25-100 lb capacity with any free-flowing substance from powder to pellets. The machine has push-button controls and is fully automatic, the manufacturer states.—E. D. Coddington Mfg. Co., 5046 N. 37th Street, Milwaukee 9, Wis.

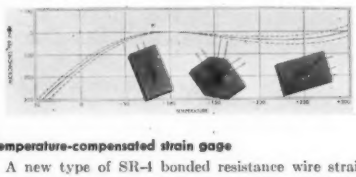


Abrasion testing

The new Taber Abraser features sturdier shafts and larger ball bearings, vacuum unit with dual suction nozzle for pickup of abradings and an electric counter for registering wear cycles. Three ranges of wheel pressures against the specimen are provided. A variety of specimen holders have been developed, such as the rimmed holder that can be partially filled with liquid sufficient to wet the specimen or completely cover it while undergoing abrasion test.—Taber Instrument Co., 111-AC Goundry Street, North Tonawanda, N. Y.

Strain recorder

A new recording SR-4 strain amplifier, the Baldwin-Sanborn recorder, which reproduces both static and rapidly changing SR-4 strain gage measurements of strains, forces, fluid pressures, displacements, vibrations and acceleration on a strip chart with rectangular coordinates has been announced by the Baldwin-Lima-Hamilton Corp. It is a direct-writing inkless instrument incorporating (1) an amplifier giving an over-all gain of at least 6000 and (2) a fast reacting D'Arsonval galvanometer in which 10 ma, representing an SR-4 gage strain of 50 micro-in. per in., for example, produces a writing arm torque of 200,000 dyne-cm and deflection of 1 cm on the chart.—Testing Equipment Dept., Baldwin-Lima-Hamilton Corp., Philadelphia 42, Pa.



Temperature-compensated strain gage

A new type of SR-4 bonded resistance wire strain gage is self-compensated for temperature variation. Principal advantage of the new gage is elimination of a second strain gage to compensate for temperature changes, thus saving application time and wiring, stated the manufacturer. The gages are available for use on Dural and steel in the form of single elements,

double elements and rosettes in two temperature ranges: 50 to 300 F and −50 to +300 F.—Testing Equipment Dept., Baldwin-Lima-Hamilton Corp., Philadelphia 42, Pa.

Perlite aggregate

An 8-page brochure on Permalite lightweight aggregate has been released by Great Lakes Carbon Corp. It is separated into two sections, one on lightweight plaster and the other on lightweight insulating concrete. Typical applications and technical data are included. The data covering mix design information includes typical properties and mix proportions by volume.—Great Lakes Carbon Corp., Building Products Division, 18 East 48th Street, New York 17, N. Y.

Waterproofing

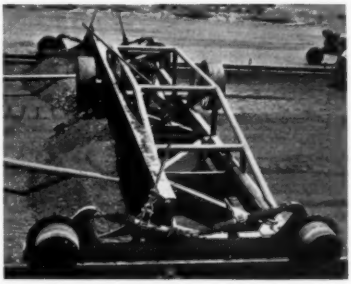
A new liquid compound, Lipton All-Seal Waterproofing, is claimed to provide waterproofing for concrete, brick, stone, tile and other surfaces. The manufacturer states that the compound lines surface voids, preventing the entrance of moisture, but does not prevent "breathing." It may be applied by brush or low pressure garden type spray.—Progressive Enterprises, 1001 N. Vermont Ave., Los Angeles 27, Calif.

Apprentice training

"Related Instruction—A Key to Apprentice Training in Construction," a booklet prepared by the Chamber of Commerce of the United States, discusses apprenticeship programs and emphasizes the role of vocational education in the related technical and supplemental instruction of apprentices. The booklet is available at 10 cents per copy.—Construction and Civic Development Dept., Chamber of Commerce of the United States, Washington 6, D. C.

Gravel plant

A new addition to the Traveler series of gravel crushing, screening and loading plants has been announced by the Universal Engineering Co. The CSE Traveler plant consists of a jaw crusher, shovel loading hopper with reciprocating feeder and trap grate, feed conveyer, one-deck inclined gyrating screen, delivery conveyor, return bucket elevator and power unit. The manufacturer states that the plant is designed to produce accurately sized material with a single crusher in a closed circuit.—Universal Engineering Corp., Cedar Rapids, Ia.



Subgrade planer

Designed to answer the demand for a heavy duty, rigid machine which will not deflect under extreme loads, it is claimed that the Ferguson subgrade planer will cut an accurate subgrade for concrete paving up to 25 ft wide. The blade will cut from 6 to 14 in. deep and is adjustable up and down by rachet jacks acting independently on each wheel. The manufacturer states that the planer is easy on forms because of its Timken bearing automotive type wheels.—Shovel Supply Co., P. O. Box 1369, Dallas 1, Texas.

Aggregate heating

Littleford Bros., Inc., report that the "Kwik-Steam" vapor generator takes only two minutes to produce steam form a cold start and less than 60 minutes to heat aggregate and water to satisfy a peak load. The manufacturer reports that when the generator is started automatic controls take over and maintain a predetermined steam pressure at all times. Sizes range from 20 to 165 boiler horsepower.—Littleford Bros., Inc., 457 E. Pearl Street, Cincinnati 2, Ohio.

SYNOPSES of recent ACI Papers and Reports

Institute papers of this JOURNAL Volume 22 are currently available at prices indicated. Please order by title and title number.

SOME OBSERVATIONS ON THE USE OF REINFORCING STEEL IN CONCRETE PAVEMENTS.....47-1

Price 35 cents.

BENGT F. FRIBERG—Sept. 1950, pp. 1-16 (V. 47)

The paper contains a few accumulated observations from a review of literature on the evolution of reinforced concrete pavement designs, with reference to various systems of reinforcement which have seen extended use. Typical structural concrete pavement failures are described. Trends in design and reinforcement practice are shown, with special reference to recent developments. Pertinent findings of a few published pavement surveys are cited, with special attention to the 900-mile Louisiana pavement survey made in 1945. The need for performance information and additional research is stressed.

TESTS OF PAPER MOLDS FOR CONCRETE CYLINDERS.....47-2

Price 35 cents.

ROBERT A. BURMEISTER—Sept. 1950, pp. 17-24 (V. 47)

Concrete test cylinders cast in a new type paper mold had a compressive strength lower than that predicted for the concrete mix used. Investigation showed that cracks and mechanical injuries to the outer shell of the concrete cylinder caused by movement of the paper stock during the first 24 hours of curing reduced the strength of the cylinder. To a lesser degree this was true also of the paraffined paper molds in common use for casting test cylinders, specimens cast in both types of paper molds showing lower strengths than test cylinders from the same mix cast in steel molds.

ADMIXTURES IN CONCRETE.....47-3

Price 60 cents.

W. T. MORAN, F. H. JACKSON, BRUCE E. FOSTER and T. C. POWERS—Sept. 1950, pp. 25-52 (V. 47)

Five papers by members of ACI Committee 212, Admixtures, are, because of their common general subject, presented together.

Various admixtures are discussed briefly as an introduction to more detailed treatment of air-entraining materials. The relative merits of admixtures and interground agents are considered. Optimum ranges of air content for different structural uses are given with particular reference to pavements.
The advantages and disadvantages of several types of admixtures used in the fabrication of various concrete products, such as building block, cast stone, pipe, cribbing and curbing, are discussed. The admixtures considered are classified into the following groups: accelerators, air-entraining agents, gas-forming agents, water repellent agents, and workability agents.
The factors affecting bleeding characteristics and workability of fresh concrete are reviewed and the effect of admixtures on these properties is assessed.
Present knowledge of admixtures in counteracting alkali-aggregate reactions is reviewed. It is emphasized that further studies may revise thinking in this field.
The effectiveness of various concrete admixtures in inhibiting the capillary flow of water and the flow of water under pressure is considered. The types of admixtures included in the discussion are accelerators, soaps, butyl stearate, finely subdivided dry materials, mineral oil, workability agents, and a miscellaneous group of proprietary compounds.

IMPROVED SONIC APPARATUS FOR DETERMINING THE DYNAMIC MODULUS OF CONCRETE SPECIMENS.....47-4

Price 35 cents.

C. E. GOODELL—Sept. 1950, pp. 53-60 (V. 47)

After a brief introduction to sonic testing and a description of commercial equipment, the apparatus built for the Michigan State Highway Department is discussed. Reliable results can be obtained by an unskilled worker with this compact equipment which has twice the frequency band spread of the usual oscillator. A wiring diagram of the sonic apparatus is included.

ANALYSIS OF THREE-DIMENSIONAL BEAM-AND-GIRDER FRAMING.47-5

Price 35 cents.

PHIL M. FERGUSON—Sept. 1950, pp. 61-72 (V. 47)

The beam-and-girder floor with some beams carried directly by columns and others supported on girders is cited as a practical problem in frame analysis that must include the torsional stiffness of the girder. Curves showing how moment coefficients vary with this torsional stiffness are developed for a few simple cases in interior panels. These show the weakness of rule-of-thumb methods.
A practical calculation form is set up for use with the moment distribution method in solving three-dimensional problems of this type.

Proposed Revision of SPECIFICATIONS FOR CONCRETE PAVEMENTS AND BASES (617-44).....47-6

Price 35 cents.

REPORT OF COMMITTEE 617—Oct. 1950, pp. 93-116 (V. 47)

New specifications incorporated include those covering air entrainment, removal of forms, premolded joint fillers and joint filling materials and method of placing reinforcement. Definitions have been added under soil foundation preparation and other parts of this section have been revised.

LINEAR TRAVERSE TECHNIQUE FOR MEASUREMENT OF AIR IN HARDENED CONCRETE.....47-7

Price 35 cents.

L. S. BROWN and C. U. PIERSON—Oct. 1950, pp. 117-124 (V. 47)

The method described for the determination of air in hardened concrete permits the examination of 6 x 8-in. and 6 x 10-in. random plane face-ground hardened concrete specimens which more truly represent the aggregate and air voids in the actual concrete than smaller specimens. The construction and use of the instruments are discussed and results of tests are given. Because of the time and equipment necessary to measure air content by means of the integrator, it is not adaptable to field use. However, as a laboratory tool it provides a means for quick and accurate determination of total air.

INFLUENCE OF THE QUALITY OF MORTAR AND CONCRETE UPON CORROSION OF REINFORCEMENT.47-8

Price 35 cents.

RACHEL FRIEDLAND—Oct. 1950, pp. 125-140 (V. 47)

In tests to determine the influence of the quality of mortar and concrete upon corrosion of reinforcement the variables studied were cement content, water-cement ratio, consistency, grading and depth of cover. The specimens, stored in moist air or exposed to weather, were tested up to the age of 2 years.
The results indicate that consistency has a pronounced effect upon the protective value of mortar and concrete, and that there appears to exist an "optimum consistency" at which the quantity of rust is practically unaffected by time. It was also found that the usual cement contents in reinforced concrete have only a limited effect upon corrosion. It is concluded that water-cement ratio does not in itself control the rate of corrosion of reinforcement.

PROPER SAND GRADING IMPROVES MASS CONCRETE.....47-9

Price 35 cents.

THOMAS B. KENNEDY—Oct. 1950, pp. 141-152 (V. 47)

Two series of concrete mixtures were designed using 6-in. traprock coarse aggregate and eight separate gradings of natural sand ranging in fineness modulus from 3.60 to 1.35. Tests were made of the plastic concrete, and specimens were cast for tests of compressive strength, resistance to freezing and thawing, and drying shrinkage. Both series of concrete mixtures had a cement content of 2.5 bags per cu yd; one had a normal air content—4½ ± 1½ percent in the portion of the mixture passing the 1½-in. sieve—and the other had a high air content—10 ± 2 percent in the portion of the mixture passing the 1½-in. sieve. Tests indicate that good durability in freezing and thawing can be obtained within the normal air content range with fineness modulus between 2.50 and 2.90. With increased air content, however, the fineness modulus range can be increased to extend from 1.58 to 3.24. Compressive strength was generally affected adversely by increased air content, but not to a serious degree. Drying shrinkage was less with normal air content mixes than with high air content mixtures. It was least when a fineness modulus of 2.52 was used, little difference being apparent between the high and normal air content mixes with this fineness modulus. The air-entraining admixture requirement increased greatly as the fineness modulus of the sand decreased. The water ratio also tended to increase with decreasing fineness modulus.

WATER-SOLUBILITY OF ALKALIES IN PORTLAND CEMENT.....47-10

Price 35 cents.

J. L. GILLILAND and T. R. BARTLEY—Oct. 1950, pp. 153-160 (V. 47)

In an effort to show correlation of soluble alkalies with alkali-aggregate reaction, the authors hydrated a number of cements for periods up to 90 days and analyzed water extracts of the ground hydrated cement. However, the correlation with expansions of mortar bars prepared with reactive aggregate was not improved by considering water-soluble alkalies rather than total alkalies.
The rate at which the alkalies become water-soluble in hydrating cement indicates that the alkali-bearing phases in cement hydrate quite readily.

NEW PRESTRESSING METHOD UTILIZES VACUUM PROCESS.....47-11

Price 35 cents.

K. P. BILLNER—Oct. 1950, pp. 161-176 (V. 47)

The method outlined here was developed to simplify prestressing of concrete to make it generally adaptable to American ways of construction. It eliminates costly

anchorages; uses large diameter wires (5/16-in. diameter now available on the market), instead of the customary 1/8-in. diameter wire, thus greatly reducing the number of wires required; prestresses all the wires in the building element simultaneously and simplifies forming. A simplified method of design calculations for prestressed concrete and the result of tests of a beam so designed are included.

PROPOSED RECOMMENDED PRACTICE FOR THE APPLICATION OF MORTAR BY PNEUMATIC PRESSURE 47-12

Price 35 cents.

REPORT OF COMMITTEE 805—Nov. 1950, pp. 185-196 (V. 47)

This proposed ACI Standard presents briefly the advantages and disadvantages of pneumatically-placed mortar and establishes recommended practices for placing and mixing shotcrete, qualifications and duties of workmen, preparation of surface before shotcreting, reinforcing, sequence of application, and other items involved in good shotcreting.

DETERMINING OPTIMUM CROSS SECTIONS FOR PRESTRESSED CONCRETE GIRDERS.....47-13

Price 35 cents.

FRED J. UZIEL—Nov. 1950, pp. 197-212 (V. 47)

General solutions are presented for selecting economically optimum cross sections for prestressed concrete flexural members. Such a solution for simply-supported girders of rectangular cross section, for any span and load, assuming the wires prestressed after setting of the concrete and full dead weight to act during the prestressing operation, leads to a design procedure which, in the case considered, is extremely simple and rapid to apply. It also permits comparisons and studies related to the critical cross section to be made in a more general and conclusive manner. For cross sections other than rectangular, the number of variables making the solution more indeterminate in nature, a similar direct solution is not obtainable. However, a rapid way of obtaining the minimum areas of concrete and steel required is suggested.

STEAM CURING PROTECTS WINTER CONCRETING.....47-14

Price 35 cents.

C. O. CRANE—Nov. 1950, pp. 213-216 (V. 47)

The use of live steam for protecting newly placed concrete from freezing weather and for providing initial curing has resulted in excellent concrete in the Enders Dam spillway at no greater cost than less desirable dry heating methods. This brief paper describes in detail the methods used.

ANALYSIS OF CONTINUOUS CIRCULAR CURVED BEAMS.....47-15

Price 35 cents.

BECLA VELUTINI—Nov. 1950, pp. 217-228 (V. 47)

Continuous circular curved beams can be analyzed easily by the moment distribution method if both bending and torsional end couples are considered. Formulas and tables are presented for circular curved beams of constant cross section that give the relations between the end moments and end torques and the corresponding rotations of the end sections. A proposed method of procedure is illustrated in which the bending end couples are kept separate from the torsional couples. The mathematical operations are not difficult as the convergence is rapid, but attention must be paid to the sign convention which must be definite and easy to apply.

SHEAR RESISTANCE OF TILE-CONCRETE FLOOR JOISTS.....47-16

Price 35 cents.

J. NEILS THOMPSON and PHIL M. FERGUSON—Nov. 1950, pp. 229-236 (V. 47)

Tests on certain types of tile-concrete joists indicate that the tile webs are more effective in resisting diagonal tension than is indicated by the current ACI Building Code specification. Stagger of tile joints appears to be unnecessary, since they do not seem to be planes of weakness insofar as diagonal tension is concerned. The tile reduces the deflection of the joist.

SOLUTION OF DIFFICULT STRUCTURAL PROBLEMS BY FINITE DIFFERENCES.....47-17

Price 35 cents.

ALFRED PARME—Nov. 1950, pp. 237-256 (V. 47)

Finite differences can be applied to the solution of those structural problems in which the physical relationships are expressed as a differential equation. Essentially, the technique employed consists of replacing the derivatives of the differential equation by its central difference equivalent. The problem is thus reduced to the simple task of solving a system of simultaneous linear algebraic equations. The numerical computation involved in the procedure is considerably reduced by two devices. First, the number of equations necessary to attain sufficient accuracy is reduced by an evaluation of the error introduced in substituting central differences for derivatives. Secondly, the solution of simultaneous equations is speeded by a systematic rapid tabulation of easily determined values. The procedure is applied to the design of a sheet pile wall, elliptical dome and skewed bridge to illustrate the scope and simplicity of the method.

Proposed Revision of BUILDING CODE REQUIREMENTS FOR REINFORCED CONCRETE (ACI 318-47).....47-18

Price 35 cents.

REPORT OF COMMITTEE 318—Dec. 1950, pp. 269-276 (V. 47)

Proposed changes decrease the allowable bond stress in plain bars (including the old types of deformed bars) and increase the allowable bond stresses for the new types of bars over those previously allowed for the old types. Top bars, those having more than 12 in. of concrete under them, are assigned lower bond stresses than bars in other positions. All plain bars must be hooked, which corresponds to special anchorage under the old provisions. The new bars develop sufficient anchorage by bond alone to correspond to special achorage with the old type bars. Consequently, all bars under the new provisions correspond to those with special anchorage under the old provisions.

CURING CONCRETE PAVEMENTS WITH MEMBRANES.....47-19

Price 35 cents.

C. C. RHODES—Dec. 1950, pp. 277-296 (V. 47)

To provide data to assess the advisability of continuing membrane curing of concrete pavements as an alternate method, laboratory and field tests were made to compare the effect of storage conditions on the physical properties of concrete, warping, temperature control, and strength and abrasion resistance of concrete cured with membranes and with wet burlap. A survey of pavements cured with clear membranes in spring and summer showed that cracking, when it occurred at all, was found predominantly in pavements laid in the morning hours. Comparing white-pigmented membranes with the usual wet-curing conditions in the field it was found to be efficient, practicable and about half as expensive as wet curing under the same conditions.

BLADE CHANGES IMPROVE TILTING MIXER.....47-20

Price 35 cents.

GLENWAY MAXON—Dec. 1950, pp. 297-300 (V. 47)

Recent experiments, as well as earlier studies, on changing the blading of tilting concrete mixers so as to improve the quality of the mixed concrete are described. The evolution of the blade shapes and the effect of these changes on the path of the materials through the mixer are illustrated.

PROTOTYPE PRESTRESSED BEAM JUSTIFIES WALNUT LANE BRIDGE DESIGN.....47-21

Price 35 cents.

GUSTAVE MAGNEL—Dec. 1950, pp. 301-316 (V. 47)

The tests made on a prestressed concrete beam of 154 ft 8 in. span, identical to the beams of the main span of the

Walnut Lane Bridge in Philadelphia, more than justified the adoption of prestressed concrete for the bridge. The test methods and results are described. The test beam exhibited a safety factor against cracking of about 2, which would be far lower for a reinforced concrete beam. The factor of safety against complete failure was about the same as for reinforced concrete while the deflection was less. It is possible to use prestressed concrete for beams where structural steel can not be used due to excessive deflection. The tests proved that the Walnut Lane Bridge will have an exceptional degree of safety with less weight and greater durability than would be possible with reinforced concrete, as well as being considerably cheaper than the conventional solution.

FINISHING AND CURING: A KEY TO DURABLE CONCRETE SURFACES.....47-22

Price 35 cents.

MYRON A. SWAYZE—Dec. 1950, pp. 317-332 (V. 47)

After a comparison of past and present pavement curing and finishing techniques the significance of timing and character of finishing and the timing and mode of curing are discussed. Laboratory tests are cited to show the effect of time of finishing and curing on surface durability to freezing and thawing. It is recommended that all concrete exposed to frost contain entrained air, have a low water-cement ratio and be thoroughly compacted after placing. A finishing and curing procedure is suggested which is adapted to the ambient conditions and to the hydration needs of the cement.

ECONOMY THROUGH BETTER CONTROL OF REINFORCING STEEL.....47-23

Price 35 cents.

F. TESSITOR and P. ROSEWARNE—Dec. 1950, pp. 333-340 (V. 47)

Difficulties and troubles encountered through the use of reinforcement steels not in strict accordance with present-day specifications are discussed. Data are presented to show the possibility of alleviating the situation for engineers, producers and contractors. Simplification of materials requirements, liberalization of code requirements to permit hot bending of bars and permanent identification of grade of steel would, in large measure, permit designers to apply reinforcement to structures in a more effective manner.

Proposed Revision of MANUAL OF STANDARD PRACTICE FOR DETAILING REINFORCED CONCRETE STRUCTURES (ACI 315-48).....47-24

Price 35 cents.

REPORT OF COMMITTEE 315—Jan. 1951, pp. 349-352 (V. 47)

Changes are proposed in bar designations to conform to the numbered designation of the U. S. Department of Commerce and in all drawings to agree with new bond values and anchorage details for new-style deformed bars. Editorial changes in the text of the Standard are outlined.

COARSE-GROUND CEMENT MAKES MORE DURABLE CONCRETE.....47-25

Price 35 cents.

HAROLD W. BREWER and RICHARD W. BURROWS—Jan. 1951, pp. 353-360 (V. 47)

A test procedure for mortar ring specimens is described. Rings containing coarse-ground cement shrunk less and showed greater resistance to freezing and thawing and to outdoor exposure than those containing fine-ground cement. These laboratory tests indicate that coarse-ground cement produces more durable concrete than fine-ground cement.

SIMPLE EQUIPMENT ECONOMICALLY EXPLORES PRESTRESSING..47-26

Price 35 cents.

MARVIN L. MASS and JACK R. JANNEY—Jan. 1951, pp. 361-364 (V. 47)

A tubular grip utilizing type metal to hold the prestressing wires is described. Other equipment includes a framework against which the prestressing force is applied and adjustable formwork for experimental beams. The equipment is so simple that it can be easily and inexpensively built to permit study of prestressed reinforced concrete in the small laboratory.

LABORATORY TESTS OF SPACED AND TIED REINFORCING BARS....47-27

Price 35 cents.

WILLIAM T. WALKER—Jan. 1951, pp. 365-372 (V. 47)

Tests of beam and pull-out specimens containing spaced and tied reinforcing arrangements indicated that little or no advantage would be obtained by spacing deformed reinforcing bars at splices. Pull-out tests showed that for deformed bars, placed vertically, in which interlocking of lugs could take place there was a slight increase in strength due to tying the bars. Beam tests showed no significant difference between spacing and tying.

Twelve beams containing three different types of deformed bars either spaced or tied and extending the entire length of the specimens were tested. The sixty vertical pull-outs tested also contained three different types of deformed bars. Half of these were made with the bars spaced and half with the bars tied together. The reinforcing arrangement brought only one bar out of the specimens for pulling; the other was allowed to bear on the base plate.

Spacing of the bars in both beams and pull-outs conformed to the joint code minimum spacing requirement of 2½ diameters center to center with a minimum clear distance of 1½ times the maximum size of coarse aggregate.

FINISHING AIR-ENTRAINING CONCRETE PAVEMENTS.....47-28

Price 35 cents.

CHARLES W. ALLEN—Jan. 1951, pp. 373-376 (V. 47)

Based on a survey of state highway department practices, common difficulties in finishing air-entraining concrete pavements are enumerated. Practices in adjusting transverse finishing machines are given and revised finishing practices are discussed. Delayed finishing, as specified in several states, is considered in connection with the riding qualities of the finished surfaces. On the basis of experience in Ohio it is concluded that smooth pavements can be built of air-entraining concrete without delayed finishing.

EFFECTS OF LATERAL LOADS ON ARCHES.....47-29

Price 35 cents.

JAMES P. MICHALOS—Jan. 1951, pp. 377-388 (V. 47)

The effects of lateral loads on arches and the possible magnitude of these effects are considered. An influence table and influence lines are presented for moments and shears in unbraced parabolic arch ribs of constant cross section. These values are for several ratios of rise to span and several ratios of bending to torsional stiffness. The effect of haunching is studied and its possible importance is assessed. Procedures are presented for drawing approximate and exact curves of moments for unbraced arch ribs and for arch ribs braced with struts normal to the ribs. Numerical studies are included.

GETTING MORE FOR OUR CONCRETE DOLLAR.....47-30

Price 35 cents.

I. E. MORRIS—Jan. 1951, pp. 389-396 (V. 47)

A concrete slab with alternating horizontal elements connected by sloping elements is described and its design is worked out. The system is intended primarily for roofs where an exposed ceiling is desired for economy. It may be used to span considerable distances, either as a simple span or as a series of continuous spans. The members are designed to use both concrete and steel to maximum advantage.

FACTORS INFLUENCING CONCRETE STRENGTH.....47-31

Price 35 cents.

WALTER H. PRICE—Feb. 1951, pp. 417-432 (V. 47)

The effect of mix proportions, type and brand of cement, availability of moisture for curing, accelerators and curing temperatures on the rate and potential strength development of concrete are discussed. The influence of rate and frequency of load applications, dimensions of test specimens and lateral restraint on the indicated strength are also discussed, and information is furnished on the variations in strength which might be expected on a typical job. Compressive, tensile, flexural, bond and shearing strengths are compared, and the strengths of control cylinders are compared with the strengths of cores drilled from structures at later ages. Information is also furnished on strength loss from freezing and thawing and alkali-aggregate expansion.

SONISCOPE TESTS CONCRETE STRUCTURES.....47-32

Price 35 cents.

E. A. WHITEHURST—Feb. 1951, pp. 433-444 (V. 47)

The Soniscope, an instrument which measures group velocities through as much as 50 ft of concrete, was used for the field testing of 13 bridges, one navigation lock, 14 dams and five highway pavements in 12 states. Repeated tests permitted study of changes in the condition of the concrete and the development of group velocities indicating the condition of the structure. The value of the results increases with knowledge of the materials, mix design, method of placement and other characteristics of the structure being tested.

EFFECTIVE SEALING OF CONCRETE PAVEMENT JOINTS.....47-33

Price 35 cents.

H. F. CLEMMER—Feb. 1951, pp. 445-448 (V. 47)

The development of cork, metal and rubber joint sealers is traced and work with thermoplastic and cold-pour joint filling compounds is described. The importance of proper technique of preparing and placing the newer materials is emphasized.

ENTRAINED AIR SIMPLIFIES WINTER CURING.....47-34

Price 35 cents.

JOSEPH J. SHIDELER, HAROLD W. BREWER and WILBUR H. CHAMBERLIN—Feb. 1951, pp. 449-460 (V. 47)

The investigation described was undertaken to determine the winter protection required to protect air-entraining concrete from damage by freezing. Cylinders made with Types II and V cement and various percentages of calcium chloride were cured at temperatures ranging from 10 to 70 F and tested for strength at ages ranging from 3 to 180 days. The resistance of these concretes to accelerated freezing and thawing was compared. The results indicate that the amount of winter protection, as presently specified by the Bureau of Reclamation, can be reduced when air-entraining concrete is used.

PRECAST CONCRETE CONSTRUCTION IN CANADA.....47-35

Price 35 cents.

OTTO SAFIR—Feb. 1951, pp. 461-468 (V. 47)

Precast concrete construction techniques as applied to a warehouse, parking garage and retaining wall project are described. Beams and columns for the precast concrete frames of a warehouse were cast flat in multiple forms on the ground slab. Cold weather required special precautions in mixing, placing and curing the concrete. In a parking garage, precast elements were combined with cast-in-place members to form an essentially monolithic structure. A retaining wall 25-ft high is of precast counterforts and 8-in. wall planks. Principal features of the job were the complicated shape of the counterforts and the weight of the completed units.

FACTORS IN PRESTRESSED GIRDER DESIGN.....47-36

Price 35 cents.

M. FORNEROD—Feb. 1951, pp. 469-480 (V. 47)

Following a general description of the Walnut Lane Bridge the stages of loading of the prestressed concrete girders and the sequence of construction operations as they affected the design are discussed. The owner's stress limitations are listed and longitudinal bending in girders, transverse bending in stiffener diaphrams and shear and principal tensile stress in the girder are considered. Safety factors are considered for various loading conditions.

For more than 30 years the design and building of concrete and reinforced concrete structures have been guided by the work of Joint Committees.

Recommended Practice and Standard Specifications for Concrete and Reinforced Concrete

This book (published 1940) represents the ten-year work of the third Joint Committee, consisting of affiliated committees of the American Concrete Institute, American Institute of Architects, American Railway Engineering Association, American Society of Civil Engineers, American Society for Testing Materials, Portland Cement Association.

Recommended Practice

- Materials
- Proportioning, mixing, curing and testing
- Forms and placing
- Waterproofing and protective treatments
- Surface finishes
- Design

Standard Specifications

- Materials
- Proportioning
- Measurement
- Mixing
- Curing
- Field tests
- Forms and placing
- Details of construction

$1.50 (to ACI Members—$1.00)

AMERICAN CONCRETE INSTITUTE

18263 W. McNichols Rd.
Detroit 19, Mich.

An essential ingredient of concrete is cement.
An essential aid to concrete structural design is the—

Reinforced Concrete Design Handbook

By ACI Committee 317

$2.00 —to ACI Members, $1.00

● Whether you are an established designer or a student learning the fundamentals, the Reinforced Concrete Design Handbook takes the chore out of design problems.

● Tables cover as large a range of unit stresses as may be met in general practice. BESIDES—the design of members under combined bending and axial load has been reduced to the same simple form as is used in the solution of common flexural problems. Cut down design time by using the numerous tables and diagrams provided in the R/C Design Handbook—an ACI "best seller" published cooperatively by ACI, Portland Cement Association, Concrete Reinforcing Steel Institute and Rail Steel Bar Association.

45 examples illustrate design procedures and use of the tables and diagrams . . . . .

Design of flexural members
Investigation of flexural members
Design of stirrups
Columns—concentrically loaded
Columns—eccentrically loaded
Design of footings
Determination of moments in building frames

45 tables and diagrams . . . . .

Coefficients (K, k, j, p) of rectangular sections
Resisting moments of rectangular sections
Areas and perimeters of bars
Minimum web depths
Coefficients (c) for compressive reinforcement for rectangular and T-sections
Spacing of stirrups
Spiral columns—size and pitch of spirals
Allowable stress on eccentrically loaded columns
Standard styles of welded wire fabric
Moments in beams with fixed ends
Coefficients for bending moments in building frames
. . . . . and many others

R/C Design Handbook
$2.00 (Nonmember price) (ACI Member price) **$1.00**

American Concrete Institute **18263 W. McNichols Rd.** **Detroit 19, Mich.**

DISCUSSION

Discussion closed January 1, 1951

Sept. Jl. '50

Some Observations on the Use of Reinforcing Steel in Concrete Pavements—Bengt F. Friberg

Tests of Paper Molds for Concrete Cylinders—Robert A. Burmeister

Admixtures in Concrete—W. T. Moran, F. H. Jackson, Bruce E. Foster and T. C. Powers

Improved Sonic Apparatus for Determining the Dynamic Modulus of Concrete Specimens—C. E. Goodell

Analysis of Three-Dimensional Beam-and-Girder Framing—Phil M. Ferguson

Discussion closed February 1, 1951

Oct. Jl. '50

Proposed Revision of Specifications for Concrete Pavements and Bases—Committee 617

Linear Traverse Technique for Measurement of Air in Hardened Concrete—L. S. Brown and C. U. Pierson

Influence of the Quality of Mortar and Concrete upon Corrosion of Reinforcement—Rachel Friedland

Proper Sand Grading Improves Mass Concrete—Thomas B. Kennedy

Water-Solubility of Alkalies in Portland Cement—J. L. Gilliland and T. R. Bartley

New Prestressing Method Utilizes Vacuum Process—K. P. Billner

Discussion closes March 1, 1951

Nov. Jl. '50

Proposed Recommended Practice for the Application of Mortar by Pneumatic Pressure—Committee 805

Determining Optimum Cross Sections for Prestressed Concrete Girders—Fred Uziel

Steam Curing Protects Winter Concreting—C. O. Crane

Analysis of Continuous Circular Curved Beams—Becla Velutini

Shear Resistance of Tile-Concrete Floor Joists—J. Neils Thompson and Phil M. Ferguson

Solution of Difficult Structural Problems by Finite Differences—Alfred Parme

Discussion closes April 1, 1951

Dec. Jl. '50

Proposed Revision of Building Requirements for Reinforced Concrete—Committee 318

Curing Concrete Pavements with Membranes—C. C. Rhodes

Blade Changes Improve Tilting Mixer—Glenway Maxon

Prototype Prestressed Beam Justifies Walnut Lane Bridge Design—Gustave Magnel

Finishing and Curing: A Key to Durable Concrete Surfaces—Myron A. Swayze

Economy Through Better Control of Reinforcing Steel—F. Tessitor and P. Rosewarne

Discussion closes May 1, 1951

Jan. Jl. '51

Proposed Revision of Manual of Standard Practice for Detailing Reinforced Concrete Structures—Committee 315

Coarse-Ground Cement Makes More Durable Concrete—Harold W. Brewer and Richard W. Burrows

Simple Equipment Economically Explores Prestressing—Marvin L. Mass and Jack R. Janney

Laboratory Tests of Spaced and Tied Reinforcing Bars—William T. Walker

Finishing Air-Entraining Concrete Pavements—Charles W. Allen

Effects of Lateral Loads on Arches—James P. Michalos

Getting More for Our Concrete Dollar—I. E. Morris

ALPHABETICAL LIST OF ADVERTISERS

(Page numbers refer to News Letter)

Autolene Lubricants Company....21
Baldwin-Lima-Hamilton Corporation....18, 19
Besser Manufacturing Company....23
Bethlehem Steel Company....11
Chain Belt Company.... 6
Chicago Pneumatic Tool Company....25
L. Coff....32
Dewey and Almy Chemical Company....15
Electric Tamper and Equipment Company....28, 29
Granco Steel Products Company.... 7
Inland Steel Company....17
Intrusion-Prepakt, Inc.... 2
The C. S. Johnson Company....26
Kelley Electric Machine Company....12
Lone Star Cement Corporation....Inside front cover
Raymond Concrete Pile Company....33
Solvay Sales Division, Allied Chemical and Dye Corporation.... 9
The Techkote Company....31
The Vannier Company, Inc....13
Charles R. Watts and Company.... 8

The Institute assumes no responsibility for the claims of advertisers. The advertiser is made responsible in the belief that his place in the field will be determined by the public's ultimate measure of his exercise of that responsibility.

Discussion closes June 1, 1951

Feb. Jl. '51

Factors Influencing Concrete Strength—Walter H. Price

Soniscope Tests Concrete Structures—E. A. Whitehurst

Effective Sealing of Concrete Pavement Joints—H. F. Clemmer

Entrained Air Simplifies Winter Curing—Joseph J. Shideler, Harold W. Brewer and Wilbur H. Chamberlin

Precast Concrete Construction in Canada—Otto Safir

Factors in Prestressed Girder Design—M. Fornerod

Title No. 47-37

Precast Concrete Offers Protection Against Atomic Blast*

By ARSHAM AMIRIKIANt

SYNOPSIS

Precast concrete is an excellent means for providing protection against atomic blast. The technique is suitable for new construction and is adaptable to existing structures by providing them with a protective shell. Readily-assembled framing elements can be prefabricated at regional plants and stored or stockpiled at various points for immediate use in an emergency.

For design purposes, in this paper, shelters are divided into three groups in relation to their proximity to a probable target. Suggested arrangements and main details of assembly of a number of types of precast framing suitable for each group are presented. A general discussion of the protection problem, the needed weapons data and design criteria are also given.

INTRODUCTION

The possibility, or perhaps the probability, of atomic warfare has created universal concern for personal safety. Heretofore, in this country, having escaped the direct ravages of two world wars, little serious thought was given to structural protection against weapons of devastation. That feeling of relative security of the home-front, predicated on waging wars away from our shores, has now waned. Fast-changing conditions of a world in turmoil, as evidenced by the Korean war, have finally brought home the grim prospect of another conflict whose new weapons may endanger everyone's life—irrespective of boundaries or location.

The new situation has resulted in a corresponding change in domains of responsibility. In past wars, before the atomic bomb, only a limited category of structures considered as vital for military operations were protected. The task was accordingly the responsibility of the military authorities. The situation is now changed in two important respects. First, the destructive effect of the atomic bomb is not confined to a single objective or structure. While it may be aimed at a specific target, the neighboring structures will also suffer damage in varying severity depending on their proximity to the center of explosion. Second, as a corollary, the probabilities of damage to any structure in a community are greatly increased. Thus the scope of the needed protective work is tremendously enlarged, and the responsibility for the major part of the task is necessarily relegated to civil defense authorities.

As yet no clear-cut policies have been adopted by the various local and public agencies regarding protective structures against atomic blast. The newly created Federal agency for civil defense, responsible for coordination and guidance of all efforts in this direction, faces unprecedented problems of extreme complexity. One of the major problems is the provision of shelters, to be used either as an initial shield against blast and radiation at time of attack or as a supplementary facility in an emergency following an attack.

*Presented at the ACI 47th annual convention, San Francisco, Calif., Feb. 21, 1951. Title No. 47-37 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 22, No. 7, Mar. 1951, *Proceedings* V. 47. Separate prints are available at 50 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than July 1, 1951. Address 18263 W. McNichols Rd., Detroit 19, Mich.

The opinions and ideas expressed are those of the author and not those of the Dept. of the Navy.

†Member American Concrete Institute, Head Designing Engineer, Bureau of Yards and Docks, Dept. of the Navy, Washington, D. C.

Photo p. 497, H. Armstrong Roberts.

While the need for shelters is accepted in principle by most of the cognizant local authorities, there is no common understanding regarding either the extent or the types of structures to be provided. The problem is further complicated by factors of procurement. Success of the program will depend not only upon the availability of means for its execution, in men, money and materials, but also on the important element of time. On the basis of these considerations, thin-shell precast concrete is a practical method of construction of the needed shelters.

Standardized designs of framing and methods and procedures of fabrication will unquestionably simplify the task of construction. However, no single design can be evolved which would meet varying requirements with equal efficiency. Therefore, various types of framing are described to provide a choice under differing conditions. The presentation is made schematically, to suggest arrangements and principal details, omitting actual dimensional characteristics which will vary, obviously, with the size of the structure, the manner of exposure, the intensity of the anticipated pressures and the desired degree of protection.

DESIGN CRITERIA

Scope of protection

A clear definition of protection against a weapon is difficult to state. According to one concept, it is a means of assuring complete safety against all the effects of an assumed weapon. To convey this idea, the intended degree of protection is further qualified by the word "full" or "complete." For the engineer charged with the required design, such a broad and all-inclusive definition has little appeal or justification. He will admit that his design, at best, may be adequate for the given weapon only under certain combinations of conditions and assumptions. He knows also that the factors which affect the efficacy of a weapon can easily change to create a critical condition far beyond the range anticipated in design.

According to another view, it is partial security against certain specified aspects of a given weapon. This second definition, predicated on more practical considerations, rules out an unqualified assurance of safety and admits the possibility of danger as a calculated risk. Most engineers experienced with the limitations of protective design consider this a reasonable approach to the problem. The writer's recommendations given here for a practical structural design to resist atomic blast conform with this understanding.

There is a great deal of information available regarding the effects of the A-bomb. Theoretically, it is an irresistible weapon. That is to say, no structure designed in accordance with our present knowledge of structural behavior could escape destruction--if made its sole target. No material could resist the fusing and vaporizing effects of the tremendous heat and pressures generated at or near its center of explosion. Fortunately, however, it is so critical and expensive an item that it cannot normally be expended

to destroy a single object. Unlike conventional weapons, devised to cause severe local damage, the objective of the A-bomb is to distribute its destructive power over the largest possible area. This is the condition of optimum efficiency, corresponding to a case where the total aggregate damage to the area of attack is a maximum. To obtain this condition, the bomb is burst at a considerable height above the target area. For the presently known type of bomb, the required height for an air burst of optimum efficiency is estimated as 2000 ft above ground. Owing to this height, the resulting pressures on the

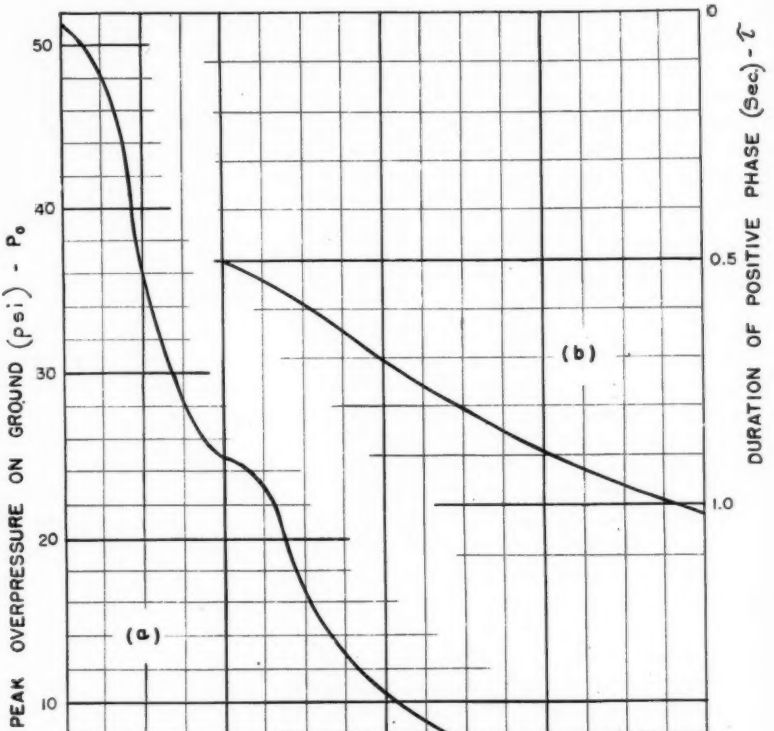


Fig. 1—Pressure and time characteristics of a 20-kilo-ton bomb exploded 2000 ft above ground. a—Peak overpressures on ground. b—Duration of positive phase

exposed structures within the affected area are greatly reduced, and a reasonably adequate protective design is correspondingly simplified.

The atomic bomb considered in design has an explosive energy equivalent to that of 20,000 tons of TNT: hence, it is referred to as a 20 kilo-ton bomb. The two bombs exploded over Hiroshima and Nagasaki in Japan were in this category. When such a bomb is exploded at an altitude of 2000 ft, the resulting pressure at ground level directly below the center of explosion (ground zero) will have an intensity of 51 psi. The pressure intensities at other distances from this reference point are given by the curve in Fig. 1(a).* These are the pressures which occur just above the ground and include the effects of the incident explosion wave and its reflection from the ground. At approximately 2000 ft from ground zero, the incident and reflected waves combine to form a horizontal shock front which is again reflected when encountered by a structure in its course. The distribution of pressures on a rectangular building resulting from such an encounter is shown in Fig. 2. As the shock wave strikes the front wall of the building at the most unfavorable angle (about 70° from normal), by reflection the peak overpressure (p_o) on that face rises to an intensity three times that of the initial value. Immediately after it reaches maximum intensity, the pressure drops first sharply to the initial value then at a reduced rate to atmospheric. The modes of variation of pressures on the roof and the rear wall of the building, also

*From "The Effects of Atomic Weapons," by Los Alamos Scientific Laboratory, June 1950.

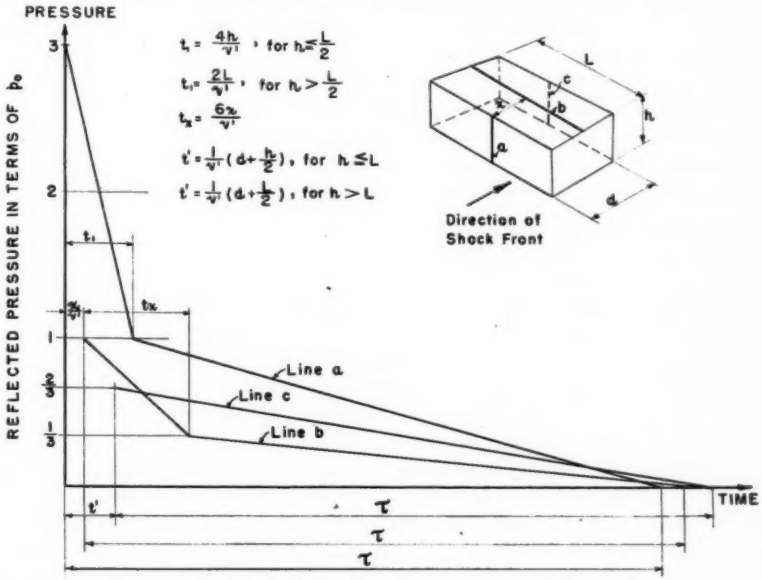


Fig. 2—Pressure distribution on a rectangular building subjected to a shock wave

plotted against time, are as shown and defined on the diagram. The duration of the positive phase for various distances is given in Fig. 1(b).

This information is the basic data which may be used in designing protective shelters against the atomic bomb. The structure contemplated is one which will assure a reasonable degree of personal safety for its occupants against the blast and radiation effects of a 20 kilo-ton bomb, assumed to explode in the air at an altitude of 2000 ft. For purposes of design, protective structures may be classed in three groups, depending on their proximity to the assumed ground zero or the probable target. The corresponding areas may be defined as follows: (1) an area of ¾-mile radius surrounding the target; (2) an area extending from ¾-mile radius to 1-mile radius; and (3) an area outside the circle of 1-mile radius. The boundary of the first area is established by the critical range of nuclear radiation. The division of the other two areas is predicated on practical considerations of framing.

Basis of design

In conventional design, the concept of resistance is predicated primarily on elastic behavior. In designing a framing to sustain a given load, the elements are so proportioned that the resulting computed strains remain within the elastic range of the materials of construction. Under ordinary load conditions, such as dead, live, snow and wind, the additional strength which may be developed in a member or framing beyond this range of strain is relatively a small percentage of the total resistance. This is not true, however, under blast loading, where the duration of this pressure is very short and during which the intensity rises sharply to a maximum and then drops to zero. In this case the greater part of the needed strength in the resisting member may be developed by plastic yielding. This is a simple corollary of the principle of work or energy, which may be stated thus: For a member of a given mass, the greater the deformation due to an impulse loading, the larger its resistance or energy absorptive capacity.

Concept of resistance

This concept of deformational resistance may be illustrated by a simply-supported beam under uniform loading (Fig. 3). The reinforcing of the member is assumed to be continuous and anchored to the fixed supports at the ends. When the unit load q is gradually increased in intensity, the beam will undergo three stages of deformation. In the first stage (Fig. 3 (b)) the behavior is first elastic then, as yielding starts at the center of the span, partly plastic. The load-deformation relation for this stage is given by the ordinary deflection expression used in conventional design. In the second stage (Fig. 3 (c)) the action is mainly plastic. During this stage, the reinforcing, having already reached its yield stress in developing the full resisting moment at the center of the span, will continue to elongate—without an appreciable increase in the imposed loading. As a result, the concrete will crack and spall extensively, causing gradual loss in the resisting moments throughout the span, and the reinforcing will begin to act as a cable. In the

third stage (Fig. 3 (d)) the deformation is entirely plastic. During this last stage, the reinforcement will continue to stretch, similar to a parabolic cable under a uniform load of increasing intensity. The load-deflection relation corresponding to the three stages of deformation is shown diagramatically in Fig. 3 (a). Here the load q may be taken as a measure of resistance of the member against deformation. For simplicity of consideration, if the discontinuous line OBC is replaced by a straight line OC, then the resistance of the member may be assumed to increase in direct ratio to the deflections obtained during the full range of deformations prior to failure. This is a reasonable assumption which will greatly simplify the analysis of deformation of a member or framing subjected to blast loading.

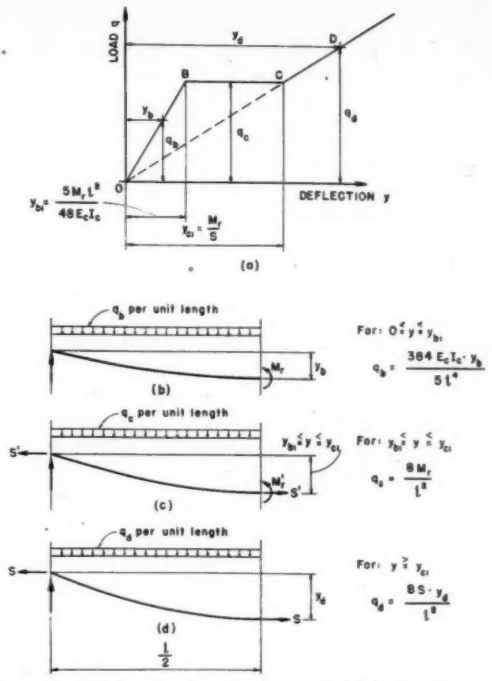


Fig. 3—Deflections of a simply-supported beam under a uniform loading of increasing intensity

The extent of permanent deformations which may be allowed in protective design will vary with the location and importance of the structure, type and arrangement of framing, the position of the affected member and the selected criteria for acceptable damage. For an economical and practical shelter design, it will be necessary to utilize the full resistance of the critical member, or the weakest component of framing, just short of collapse. If the framing is so arranged that the failure of an individual member or component does not jeopardize the over-all stability of the structure, then it will be permissible to design that member for its full resistance—by allowing large deflections under blast loading. A concrete shelter so designed will suffer considerable damage when subjected to the assumed blast pressures. The anticipated damage will be extensive cracking, spalling and local fractures which may be subsequently repaired by restoring or replacing the affected parts. If it is desired to reduce the extent of induced damage, then the allowable deformations may be limited to a more conservative value, say,

3 percent elongation. For a simply-supported member, an average strain of this magnitude will result in a deflection of about one-tenth of the span.

Method of analysis

The atomic blast imposes an impulse loading on a structure. Determining strength or adequacy of framing will accordingly consist of an analysis of dynamic behavior. Two studies are required: first, an analysis of deformations of components of a framing to see that the motion of these members relative to their supports does not exceed the limits set in the design criteria; second, an analysis to assure that the structure possesses an adequate over-all stability against sliding and overturning. A suggested method of analysis is given in another paper.*

TYPES OF FRAMING

Area One

As defined above, this is the area bound by a circle of ¾-mile radius, having its center at the assumed target or ground zero of the air-burst bomb. To provide the needed protection against nuclear radiation, a covered shelter is required in this area. For this purpose, the use of a curved or shaped framing outline is advantageous. The recommended types include the following:

Type 1A—Ribbed-shell circular frame—This is the precast concrete version of the well-known "Quonset" hut. The main features of framing are shown in Fig. 4. The arch is formed by two segments, joined together at the crown by bolting, and similarly anchored to a poured or precast foundation pad. This shelter has a front barricade and a rear enclosure of flat ribbed panels. A concrete floor, either precast or poured, is optional. In cross-section, a typical arch segment is composed of a thin shell stiffened by two edge ribs and a series of cross ribs dividing the slab into approximately square subpanels. The width of the segment may vary from 2 ft to 4 ft. Recommended thicknesses for slab and earth cover, for shelters at various distances within the area, are:

Distance from target, miles	0 to ¼	¼ to ½	½ to ¾
Thickness of slab, in.	4	3	2
Thickness of earth cover at crown, in.	36	24	12

The slabs are reinforced with 2 x 2-in. wire mesh, varying from No. 6 to No. 10 gage. For a shelter of 20-ft span, a 4 x 12-in. rib section is adequate.

Type 1B—Ribbed-shell gable frame—This frame (Fig. 5) is a modification of the circular type to provide greater clearance. This advantage is obtained, however, by the use of segmental edge ribs of greater depth than those of Type 1A, because of a less favorable stress pattern. Other details and arrangement of this shelter conform to the preceding type.

Type 1C—Ribbed-shell dome—The dome (Fig. 6) is composed of a number of trapezoidal panels, arranged in conical tiers, and topped with a circular closure piece at the crown. The panels, of like size in each tier, consist of a

*Amirikian, A., "Design of Protective Structures." Paper presented at the Annual Meeting of ASCE, Chicago, Ill., Oct. 1950.

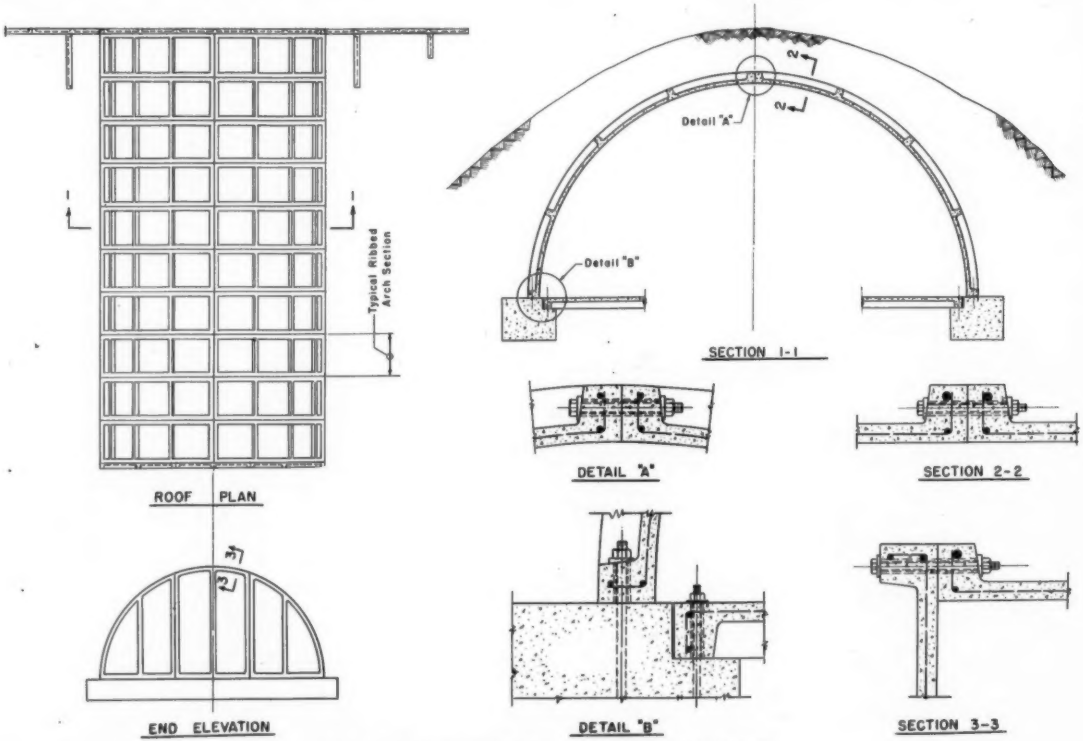


Fig. 4—Earth-covered shelter of cylindrical ribbed shell

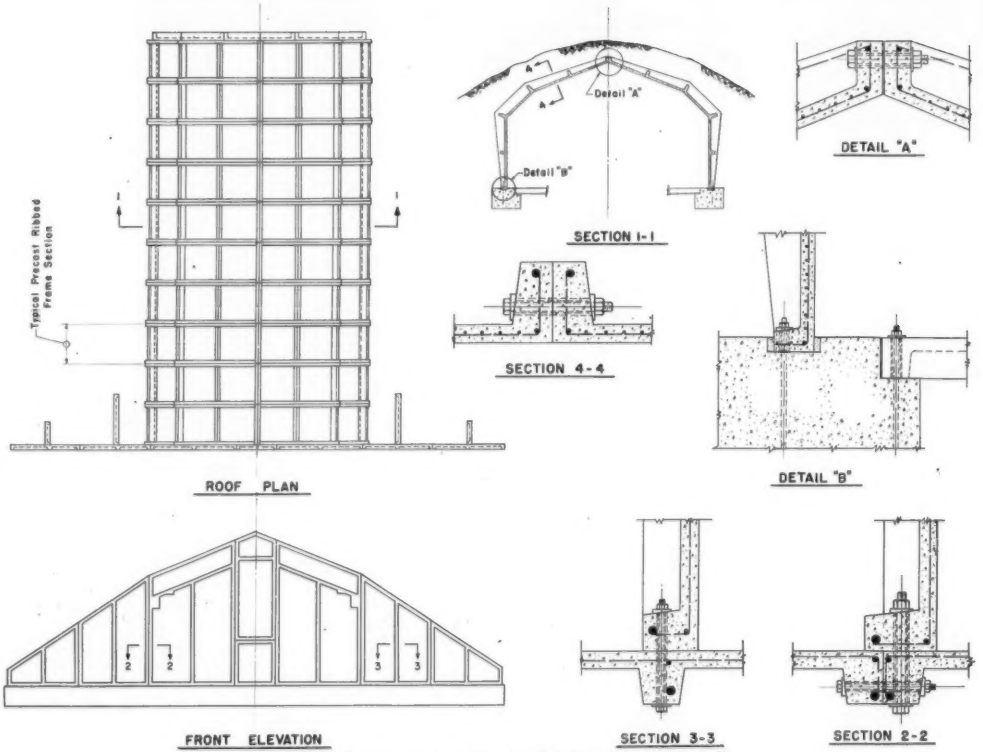


Fig. 5—Earth-covered shelter of gable-shaped ribbed shell

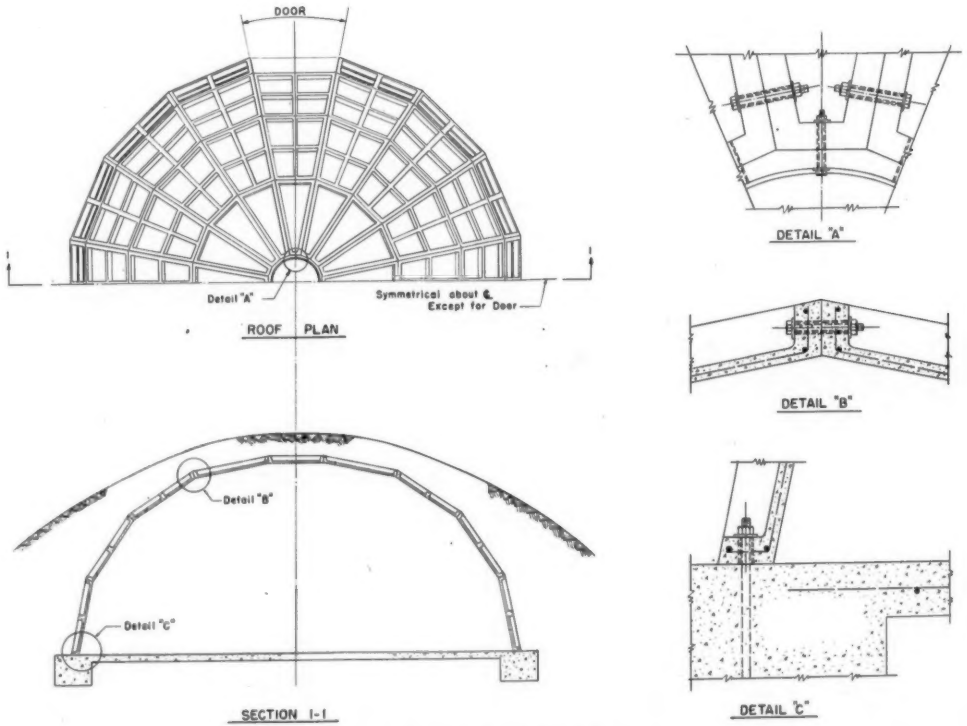


Fig. 6—Dome shaped shelter of ribbed-shell panels

skin slab and stiffening peripheral and interior ribs. Bolts through the ribs connect adjacent units. The thicknesses of the slab and earth cover are the same as those recommended for Type 1A.

Owing to the simplicity of the components of framing, this dome-shaped precast shelter can be fabricated easily and readily erected.

Area Two

This is the circular area between an inner radius of ¾ mile and an outer radius of 1 mile from the assumed target or ground zero. Owing to the much reduced rate of radioactivity at this range, no earth cover is required for shelters in this area. Accordingly, the same types of framings recommended for Area One may be used here without cover. To provide a more favorable pressure surface, all rib stiffeners of the framing panels should project inward. Likewise, a dome-shaped shelter similar to that shown in Fig. 6, but with inward-projecting ribs may be built. The shell thickness will vary from 2 in. for shelters to be erected near the outer boundary of the area to 3 in. for those near the inner boundary.

Type 2D—Cellular panel shelter—This is a rectangular shelter (Fig. 7) built of an assembly of flat panels in the roof and the walls. The panels are composed of a layer of precast concrete boxes, joined together with a poured skin slab at top and bottom faces. The reinforcement consists of bar joist trusses, placed between adjacent rows of cells, and 2 x 2-in. light-gage wire mesh in the cell faces and in the poured slabs. As shown in Sect. 2-2 of Fig. 7, the boxes are so arranged, that by the alternating sequence of placement of the open and cast faces, continuous planes of longitudinal and transverse diaphragms are formed through the depth of the panel. The thickness of the cell slab is ⅝ in. and that of the poured or shotcreted skin ⅞ in., resulting in a total thickness of 1½-in. for each face of the panel. For a panel of 20-ft span, an 8-in. cell depth is sufficient.

Area Three

This area, beyond 1 mile from the assumed target, may be considered as the zone of minimum protection. The strength and stability requirements for shelters to be built in this area can easily be met by one of the simplest and most efficient type of precast framing, panel and rib framing. The panels consist of a thin slab, not more than 1½-in. thick, stiffened with a system of peripheral and interior ribs. Typical details and arrangement of this framing element are given in an article describing their application to low-cost house framing.* In some arrangements, these panels may provide all the components of framing of the shelter. In others, where large spans are required, the panels are supplemented by supporting bents or frames. The latter members are of hollow cross section, composed of two channel-shaped elements conforming to the elevational outline of the bent. The use of this type of protective framing is illustrated by the following examples:

*Amirikian, A., "Some Problems in Structural Framing of Precast Concrete Houses," ACI JOURNAL, Mar. 1947, *Proc.*, V. 43, pp. 797-810.

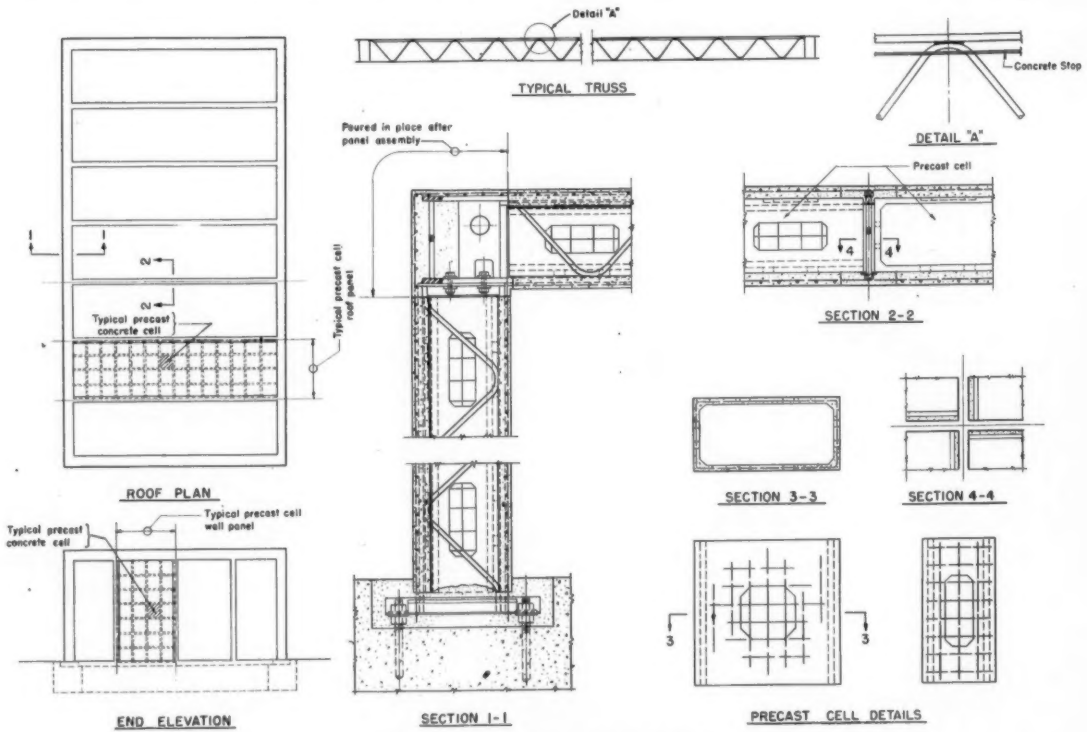


Fig. 7—Rectangular shelter of cell panels

Type 3A—Panel-and-frame building—General arrangement of framing, and main connection details are shown in Fig. 8. It consists of a series of bents supporting an enclosure of ribbed-shell panels in the walls and roof of the shelter. The system possesses considerable dimensional flexibility for adaptation to framings of large spans and clearances.

Type 3B—Panel-reinforced building—Many existing structures, lacking strength and stability to resist blast pressures anticipated in this zone, may be provided with the needed additional protection by means of an external shell enclosure. Fig. 9 shows the simplest form of this type of protection. The arrangement is adaptable to single-story brick or frame buildings of narrow width, say, not over 20 ft in span. In this case, ribbed-shell panels are attached to the brick walls and the wooden roof by through bolts. The panels form a series of rigid-frame bents which, by their attachment, also utilize the mass of the existing framing in developing resistance to blast.

Type 3C—Sheltered building—When the width of the building exceeds 20 ft, reinforcing panels need an independent supporting system, consisting of supplementary bents or frames. Essentially, this means erecting a complete frame to enclose the existing structure. In the case of a single-story, single-bay building, the exterior protective shell is similar to Type 3A (Fig. 8). Here the new enclosure can be installed without material disturbance of the framing or operational functioning of the existing building. If the building is of the industrial type, having a large floor area, then the provision of an external protective shell will necessitate cutting through the existing roof and the floor to install the needed interior columns of the supporting frame. An arrangement of this type is shown in Fig. 10. The precast framing is similar to that utilized at Mechanicsburg, Pa.*

PRODUCTION AND USE

Providing shelters for a community is a huge construction task. Its success will depend on optimum use of available materials and labor during the limited time of the existing emergency. Mass production, planned in minute detail, can be of great assistance. The proposed use of precast concrete is predicated on these critical considerations.

Fabrication

The method of producing framing elements for precast concrete shelters will vary with the number of structures and the diversity of types in each area. In most cases, the required quantities of each framing element will probably warrant planned factory production. This will require a concreting plant, a lumber mill or carpentry shop to fabricate the templates for the concrete molds, a shop for fabricating the reinforcement assemblies, casting and storage yards, and a system of weight handling equipment. To provide some further flexibility in the over-all operation, some of these activities, such as the concrete mixing plant, the preparation of the molds and the

*Amirikian, A., "Precast Concrete Storehouses," ACI JOURNAL, June 1947, *Proc.* V. 43, pp. 1097-1116.

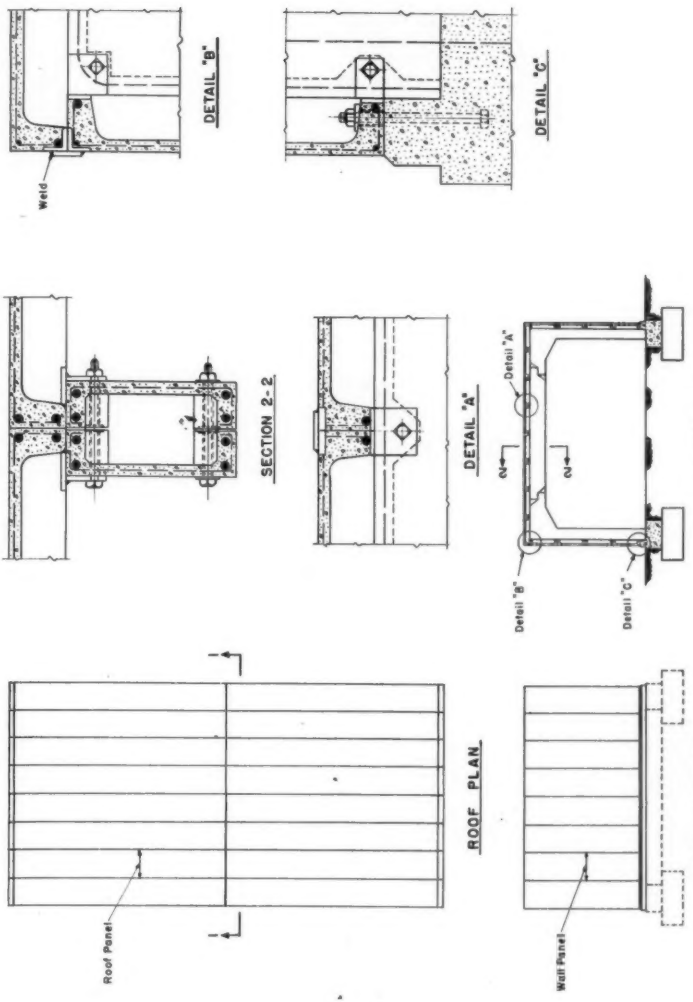


Fig. 8—Rectangular shelter of ribbed-shell panels and hollow section frames

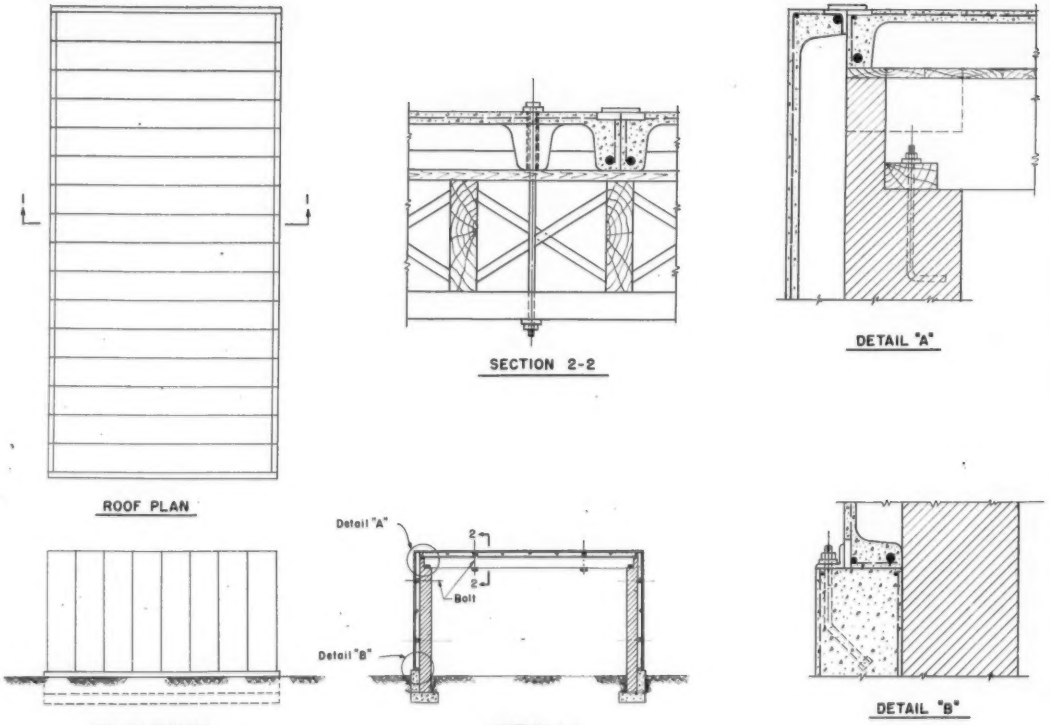


Fig. 9—Single-story brick building of small span protected with ribbed-shell panels

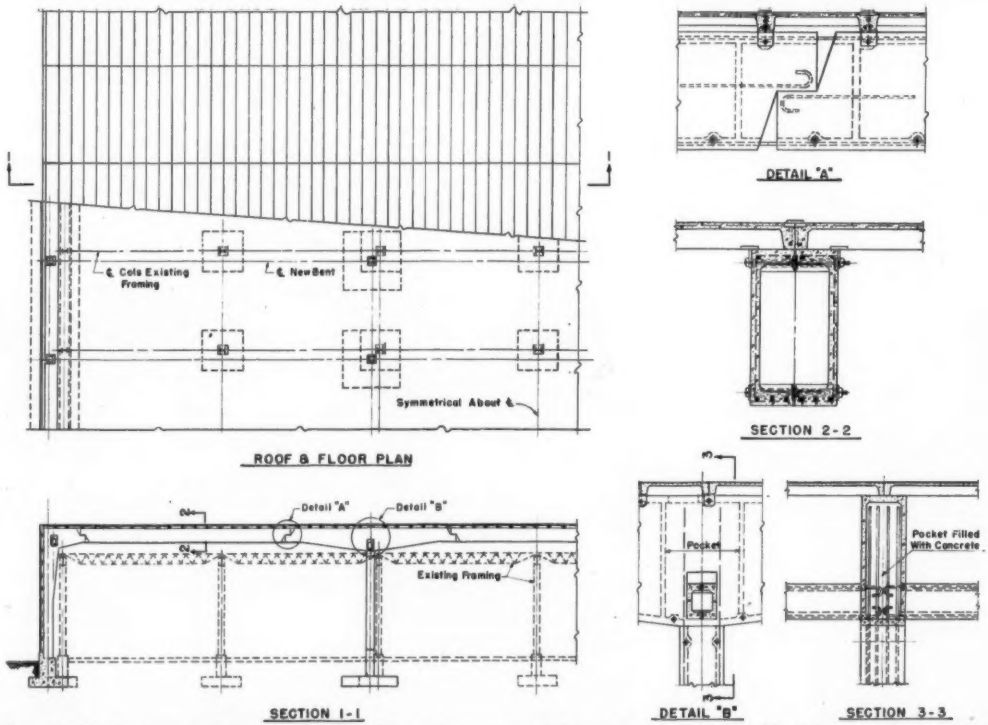


Fig. 10—Single-story, multi-bay industrial building of conventional framing protected with ribbed-shell panels supported on hollow-section precast bent frames

fabrication of the steel work, may be carried out at different plants or localities under separate managements. The various operations are described in the article on the construction of precast concrete storehouses at Mechanicsburg.*

If the number of shelters to be erected in a small community, not associated with a higher coordinating authority, is such that large-scale production methods cannot be used economically, then it becomes necessary to produce the relatively small quantities of framing elements by conventional methods of fabrication. Some phases of the work pertaining to a project of this scope are shown in Figs. 11 to 18. Fig. 11, 12 and 13 illustrate wood molds for casting ribbed-shell panels of gable and circular outline. The forms have plywood linings and fir backing strips. Fig. 14 shows a pair of form assemblies for casting small cells. Fig. 15 shows the construction of a cellular panel, consisting of a layer of cells cast between two face slabs. Fig. 16 shows the details of placement of the reinforcement and the various inserts for a ribbed-shell panel of flat rectangular outline. Fig. 17 illustrates the casting of a series of ribbed-shell trapezoidal panels used in a dome-shaped shelter.

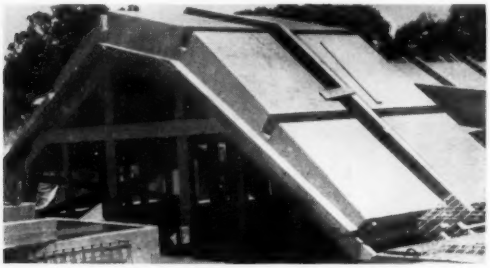


Fig. 11—Wood mold for shell frame panel with inward-projecting ribs

Fig. 12—Wood mold for shell frame panel with outward-projecting ribs

Stockpiling

When planning and procurement of shelters is assumed by cognizant authorities at state levels, then the needed vast quantities of framing material can best be secured by a central engineering agency. This agency, in turn, would apportion the fabrication work among the various construction firms within that state. All operations, would, however, conform to the plans devised at the central agency. Framing components for adopted types of structures would be stockpiled at convenient distribution centers for eventual transfer and use where needed. Framing material thus stockpiled will constitute a prudent investment which can be readily utilized in any emergency.

*Corbetta, Louis P., "Precast Concrete Warehouse Construction," ACI JOURNAL, June 1947, *Proc.* V. 43, pp. 1117-1124.

Fig. 13—Wood molds for cylindrical shell panels with interior and exterior stiffening ribs

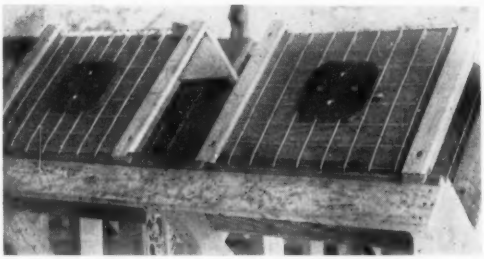


Fig. 14—Form assembly for casting cells. Note wire mesh and detail of opening inserts

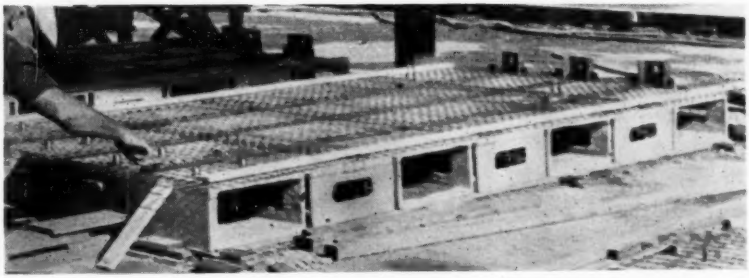


Fig. 15—Casting of cell panel. Details of assembly prior to placing top slab

CONCLUSION

The grim shadow of threatening events of an uncertain future leaves little room for complacency or optimism. Pessimism, born of a feeling of defeatism, may prove equally dangerous. While fervently hoping to be spared the horrors and devastation of another war, we can ill afford to let the present emergency run its unpredictable course without contributing our full share to the common effort of preparedness. What protective measures can be devised in the limited time? The shelter problem is a part of that challenging query. Pre-

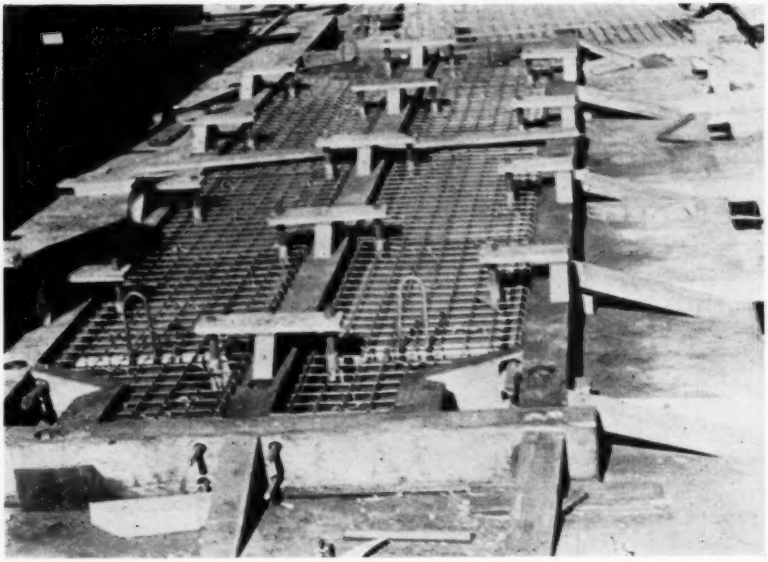


Fig. 16—Mold for rectangular ribbed-shell panel. Reinforcement assembly, bolt-sleeve holding device and lifting hooks

Fig. 17—Casting of ribbed-shell trapezoidal panel for dome frame

cast concrete can play an important role in meeting this challenge successfully. Its application presents encouraging possibilities which merit further exploration and full utilization by the construction industry.

A review of the literature on, and recent experiences with false set in portland cement with recommendations for overcoming this condition.

False Set in Portland Cement*

By R. F. BLANKS† and J. L. GILLILAND†

SYNOPSIS

False set of cement causes difficulties in mixing and placing and even though the stiffening is eliminated by job conditions, extra mixing or the addition of corrective admixtures, undesirable effects on the hardened concrete remain. It adversely affects water requirement, strength, bond between aggregate and matrix, brittleness and cracking, resistance to freezing and, thawing and air-entraining characteristics.

Correctives in the manufacture of the cement are proper cooling of mills or the use of stable calcium sulfate.

SUMMARY

On the construction job, false set (variously called premature stiffening, rubber set, gum set, or hesitation set) is evidenced by a definite stiffening or loss of consistency during mixing or shortly after. In extreme cases, the concrete may hang up in the top of the mixer drum, or stiffen in buggies, truck bodies or buckets while in transit to such a degree that picks and shovels may be required in the discharging operation. Frequently, however, concrete afflicted by false set in the cement will escape detection on the job. This may result from extended periods of mixing or agitation which break through the stiffening, restoring the concrete to nearly normal consistency. In mild cases, about the only observable effect on the concrete is a high water requirement which, of course, can be caused by many other factors. However, even though the stiffening characteristic itself be eliminated by job conditions, extra mixing or the addition of corrective admixtures, there is growing evidence that the undesirable effects on the hardened concrete remain.

Stiffening of concrete due to false set in the cement is highly unpredictable because it is affected by many factors. First of all, cement varies widely in degree of false setting. Job conditions such as temperature, length of

*Received by the Institute Jan. 12, 1951. Presented at the ACI 47th Annual Convention, San Francisco, Calif., Feb. 21, 1951. Title No. 47-38 is a part of the copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 22, No. 7, Mar. 1951, *Proceedings* V. 47. Separate prints are available at 35 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than July, 1, 1951. Address 18263 W. McNichols Rd., Detroit 19, Mich.

†Members, American Concrete Institute; Chief, Research and Geology Division, and Head, Chemical and Cement Laboratory, respectively, U. S. Bureau of Reclamation, Denver, Colo.

mixing and agitation time, method of transportation, character of mixing water, and type of aggregate may all have a profound influence on the stiffening characteristic. Regardless of the degree of concrete stiffening on the job, false setting cement undesirably affects concrete through:

1. High water requirement
2. Low strength
3. Low bond between aggregates and matrix
4. Brittleness and high cracking tendencies
5. Low resistance to freezing and thawing
6. Erratic air entraining characteristics

When concrete stiffens on the job, it delays construction, causes excessive bleeding, makes uniform control of concrete quality practically impossible, and increases handling, placing and finishing costs.

While there have been many theories propounded regarding false setting in cements, it is now generally agreed that the usual cause of the phenomonon is unstable gypsum (plaster set). The most common cause of unstable gypsum in cement is high grinding mill temperatures. Instability in gypsum is also affected by storage temperatures, aeration, and moisture.

Wherever proper precautions to cool the mills have been taken, false set has disappeared. Therefore, it must be concluded that most cases of false set can be corrected in the manufacturing process. True, some mills produce an acceptable cement even though they are operated at very high temperatures. Possibly they can be operated at these temperatures without introducing false set because the gypsum is being converted to insoluble anhydrite. Nevertheless, cooling the mills has been so completely successful that it should be given first consideration in correcting false set.

Another possible approach is to use stable calcium sulfate, either natural anhydrite, or gypsum which has been calcined to form insoluble anhydrite. Limited tests have shown that low alumina (low C_3A) cements may be adequately regulated with these materials.

THEORIES ON FALSE SET

There are two definitely different types of early set: a true, quick set, accompanied by the evolution of considerable heat; and a false set, during which little heat is evolved. Quick or flash set indicates that the cement has not been sufficiently retarded. Normally this condition can be corrected by additional retarder, although underburning is a contributing factor.

On the other hand, stiffening from false set can usually be destroyed by continuous mixing, or by remixing. False set is an elusive condition, appearing or disappearing in shipments from the same bin of cement. In severe cases, however, there is no question of its presence, or of its undesirability.

It is surprising to note that the ACI JOURNAL does not contain a single article on false set in cement. Neither Meade's *Portland Cement* (1926)[1] nor Eckel's *Cement, Limes, and Plasters* (1928)[2] mentions it. However, in 1928 Meade[3] noted that seasoning may make cements quick-setting, and that

cement heated to 400 F, or stored in silos at lower temperatures, becomes quick-setting. Further[4], he considered that some of the causes of setting difficulties are (a) lack of balance between CaO and $SiO_2 + Al_2O_3 + Fe_2O_3$, (b) lack of uniformity in burning, and (c) grinding hot clinker. Meade said that some laboratory cements ground in jar mills would be quick-setting, whereas the same clinker in a large mill would be satisfactory. He attributed the difference to the escape of moisture in the large mills. Although he described the result as quick-set, it is not unreasonable to assume that some of the quick-sets and setting difficulties of which he wrote were false set.

In 1929, several articles on false set appeared in the British publication *Cement and Cement Manufacture*. The first of these[5] noted that false set occurred on 2 minute mixing, but not on a 4 or 5 minute mix. Brandenburg[6] said that he could induce stiffening in any well-made cement by raising the finish end mill temperature to 230 F or higher. He concluded that "Evidently the entirety of the gypsum becomes converted to anhydrite, thus losing its solubility and speed of reaction efficiency as a retarder." Another contributor, "C.E.H.,"[7] felt that the false set was due to heavy charges of grinding bodies raising the temperature of the finished cement to over 338 F. He found that a suction fan and exterior-spray cooling caused the cement to become normal setting. Another contributor[8] thought that false set was due to insufficient gaging, so that each particle was not wetted. Finer cements required more mixing to wet all the particles.

The following year, Mehta[9] objected to the emphasis on the effect of grinding temperatures on false set, reporting one case in India where normal set was obtained, even though the final temperature of the cement was 300 F. He believed that false set was due to the quality of the clinker. It was at this time that Whitworth,[10] after working with cements in which false set had been induced by heating, suggested that false set was due to a portion of the cement setting separately. He found that cement showed false set even in a 1:3 cement and sand mortar.

In 1931, Anzlovar[11] proposed that the rehydration of dehydrated gypsum actually removed sufficient water to dry up the mix. He agreed with Mehta's statement that incorrect burning was a cause of false set. He believed that longer and more intensive burning produced lime compounds which were not readily hydrolyzed, allowing anhydrite sufficient time to dissolve and react. When anhydrite was used, high grinding temperatures would not result in false set. Koyanagi[12] proposed that false set could be eliminated by reducing the gypsum content to about 1 percent SO_3. Whitworth[13] disagreed with Koyanagi, and cautioned of the dangers attendant on reducing the gypsum too far. He noted[14] three transition points in heating cement which originally was slow setting. At 300 F, he found false set, which he ascribed to the hemihydrate. At slightly over 400 F, he found a true, rapid set, due to the ineffectiveness of the low solubility calcium sulfate which he believed existed at that temperature. At about 900 F, he thought that a readily soluble calcium sulfate was formed. Cement so heated had normal set.

The idea that false set was actually a drying up of the mix due to the crystallization of hemihydrate to gypsum was again proposed in 1933, this time by Schachtschabel.[15] Whitworth[16] promptly refuted the theory.

In 1934, Davis[17] described false set as applying to the initial set only, characterized by the loss of surface moisture, and the stiff, dull appearance of the neat cement pat very soon after gaging. He attributed the reaction to dehydration of gypsum during grinding, and stated that the difficulty was overcome by water-cooling the grinding mills.

A summary of the earlier literature on false set was published by Blank[18] in 1935. He cited his own experience with false set, which he attributed to grinding wet clinker. Lea and Desch's text[19] on the chemistry of cement discusses false set briefly, differentiating between false set and flash set on the basis of the amount of heat generated in the cement paste. They felt that the condition was connected with the dehydration of gypsum during grinding, but that it was favored by under- or over-burning the clinker. They explained that free lime in underburned clinkers might assist in dehydrating the gypsum during grinding.

Forsén's paper[20] on retarders and accelerators, presented in 1938 at the Stockholm Symposium, touched off a considerable discussion on false set. In discussing Forsén's paper, Bogue presented two possible causes for false set: dehydration of gypsum during grinding, and carbonation of alkalies if storage conditions are not proper. The mechanism of alkali carbonation was explained as the interaction between the alkali carbonate and calcium hydroxide, resulting in precipitation of calcium carbonate, and a decrease in available lime. Forsén stated that he had formerly believed plaster set provided a logical explanation, but he now proposed that false set was explained by the formation of $3CaO \cdot Al_2O_3 \cdot 3CaSO_4 \cdot 31\frac{1}{2}H_2O$, the trisulfate (due to the high SO_3 concentration in the liquid phase resulting from the greater solubility of hemihydrate) which did not form a protective film. On the other hand, $3CaO \cdot Al_2O_3 \cdot CaSO_4 \cdot 12H_2O$, the monosulfate, formed as an insoluble film in the presence of lime and gypsum, and retarded the set. If this were true, it would be expected that alkali-free cements would be more susceptible to false set, since Lea had found that the trisulfate was formed at 77 F in alkali-free solutions of gypsum and tricalcium aluminate, but that the monosulfate was formed in the presence of sodium hydroxide.

In Lerch's study[21] on the influence of gypsum, reported in 1946, he used heat liberation curves to examine Forsén's hypothesis that gypsum retarded the set but that plaster of paris did not. Lerch found no appreciable difference on the heat curves, so concluded the action of hemidyhrate was a plaster set.

Bogue's book[22] on the chemistry of cement (1947) repeats the causes suggested at the Stockholm Symposium, and reports that in 1933 he and Lerch observed that hemihydrate "is capable of inducing a condition of rigidity in cement pastes by itself hydrating to gypsum."

Hansen and Hunt's recent (1949) study[23] on the use of anhydrite attributed false set to partially dehydrated gypsum. They suggested that the dehy-

dration could be minimized by cooling clinker prior to grinding, cooling mills during grinding, adding small amounts of water to the mills, and by cooling the cement prior to storage.

Stadtfeld[24] attributed (1949) false set to dehydrated gypsum. He found it possible to correct an unsatisfactory cement by cooling clinker further before grinding, reducing grinding rate, and reducing SO_3 below 1.4 percent.

Barona,[25] too, decided that dehydrated gypsum was responsible (1950). False setting Mexican cements were corrected by cooling the grinding mills.

FIELD EXPERIENCES WITH FALSE SETTING CEMENT ON RECLAMATION PROJECTS

All the cement for one of the Bureau's larger dams was purchased under a specification which included a premature stiffening requirement—a penetration test using the large end of the Vicat needle, and a paste with a fixed water-cement ratio. It was required that the penetration 2½ minutes after mixing should exceed 2 mm. From the first bin sampling in May 1940, through the end of that year, the lowest penetration found was 17 mm, and most of the bin samples gave values greater than 25 mm. Starting in January 1941, over one-half of the bin samples gave penetrations below 10 mm, going down to as low as 2 mm. This break in penetration results coincides with the increased demands for cement for the defense program.

Trouble with stiffening was first reported by the project in February 1941. The concrete would stick in buckets with restricted openings, such as those used for structure work. Mass concrete for the dam did not stick in the buckets, because the buckets were designed to open fully, with no restriction to flow, but trouble was encountered at two transfer points: first, the collector cone into which the mixers dumped; and second, the hoppers on the trains that discharged into the cableway buckets. The collector cone was designed to act as a balancing reservoir between the mixers and the trains, but, with stiffening cement, 6-in. maximum size aggregate concrete with even more than maximum allowable slump stiffened enough in from 3 to 5 minutes in the collector cone to require from a few seconds to 10 minutes to jar it loose with a jackhammer vibrator. Likewise, discharge from the train hoppers was aided by vibrators, pounding on the hoppers with a sledgehammer, and banging the heavy discharge gates. As a result, the field concrete inspectors carefully noted all conditions aggravating the difficulties. They found that the stiffening usually developed within 5 minutes after mixing, and could be broken by holding the concrete in the mixer for a short period, and then remixing for ¼ to ½ minute. Since the cement had met the premature stiffening requirements in the purchase specifications, it can only be concluded that the requirements were too lenient. Parallel mixes, using another cement, yielded a concrete having none of these undesirable characteristics.

Another example occurred on a canal lining job. In addition to aggravating the contractor's difficulties when breakdown or other circumstances held up lining operations, the false set of the cement resulted in a shallow shrinkage cracking or checking at the surface before the concrete set. A typical

slab is shown in Fig. 1. The cracks varied from a few inches to a foot or two in length, were variously spaced from one to several feet apart, and were generally parallel to the centerline except on the top edges where they were normal to the edge. Usually, they did not appear on surfaces which had received a second troweling at some distance from the slip-form. This is an indication that the false set was broken by the re-troweling, after which the surface concrete proceeded to set without abnormal shrinkage. In this case, it was found that when SO_3 content was increased to 2.5 percent the stiffening characteristic of this cement was relieved and cracking was greatly reduced.

At one job in progress during 1950, the contractor was considerably inconvenienced by false set during concrete placement. Nevertheless, the workmanship was excellent. By a parallel trial the contractor found that cement from another mill did not exhibit this characteristic. The concrete no longer had a 2-in. slump loss. It did not hang up in the placing buckets, as did the concrete made from cement having false set. On this job, the contractor was furnishing the cement, and he cancelled the remainder of his original order, and purchased cement from the second mill. Obviously, that contractor solved the false set problem from his point of view.

On another recent job, the difficulty was manifested in extremely poor strength gain between 7 days and later ages, and by lack of bond between matrix and aggregate. The appearance of false set on this job coincided with a change in the brand of cement being used. Cement from Mill "A" had been satisfactory. The materials engineer at the site noted that the cement from Mill "B" had a high water requirement, an inability to entrain proper amounts of air, did not produce workable concrete and was generally unsatisfactory in both laboratory and field. It was first thought that high

Fig. 1—Shallow surface cracking on a canal lining due to false set. This type of cracking does not occur if the panel is re-troweled.

TABLE 1—FALSE SET TESTS ON CEMENTS FROM MILL "B" (As received on the project)

Date	Bin	Penetration, mm* ½ min.	Penetration, mm* 5 min.
10- 5-49	A	37	3
10-31-49	A	38	35
12- 7-49	B	40	7
12- 9-49	B	37	8
12-27-49	B	35	2
1-11-50	B	37	17
1-16-50	B	35	2
1-19-50	B	38	4
1-27-50	B	38	4
2- 1-50	B	39	7
2-11-50	C	36	31
2-17-50	C	37	6
2-18-50	C	35	2
2-20-50	C	39	36
3-10-50	C	38	22
3-15-50	C	36	29
3-16-50	C	38	3
3-23-50	D	37	30
3- -50	D	37	32

*Cements having a 5-minute penetration of less than 10 mm are rated false-setting.

placing temperatures and poor air-entraining solution were contributing to the difficulties, but later tests under ideal conditions, and with a new agent did not improve the concrete. The water requirements to produce workable mixes were 24 percent in excess of normal. As a result, the cement content was increased 0.33 barrel per cubic yard to meet workability and water-cement ratio requirements. Other theories were offered which blamed the order of batching the ingredients, the special retarding agent used to combat stiffening, and any other factors which might affect the concrete. None of these theories could be substantiated. The only consistent factor was the fluctuating water requirement of the cement, and its erratic behavior in the penetration test (described later) for stiffening. Results obtained in the stiffening test over a 6-months period are shown in Table 1. Note that there are extreme variations between cars from the same bin.

The disappearance of stiffening troubles on the job coincided with another change in the brand of cement. A recent report from the project reported that cement from Mill "C" was being loaded directly from the grinding mills onto trucks for delivery to the job, by-passing any storage at the cement plant. The cement received at the job had a temperature of 150 to 200 F. The contractor had sufficient storage to permit overnight cooling before the cement was used. False set tests were performed on each shipment of cement, and all cement was found to be acceptable.

This detailed list could be expanded to include a large number of projects, representing cement from many mills. It has been chosen to illustrate that the problem exists in the field and is not just a laboratory curiosity.

Some investigators have questioned the role of pozzolanic materials in false set problems. Swayze[26] reports that pozzolanic materials will interact with cement, and develop premature stiffening in cements which alone behave normally. He reports that Forsén found a material which was most efficient in combining with hydrated lime, but which could not be used due to the development of a virtual quick set when combined with an otherwise stable cement. Apparently, Swayze felt that this behavior could be explained by the action of alkalies liberated from the pozzolan, inasmuch as the acceleration of set by alkalies is well known. To the authors it appears that Forsén is speaking of true, quick set rather than false set, caused by alkalies, as suggested by Swayze, or due to a high water absorption by the pozzolan. False set has not been a problem on any of the Reclamation jobs where pozzolanic materials were or are being used. None of the false set examples described earlier involved pozzolans. The few laboratory test results available indicate that mild cases of stiffening may actually be somewhat relieved by the addition of pozzolans, if the water content of the mix is adjusted for any increased requirement due to the pozzolan. On the other hand, if the water content is not adjusted, pozzolans having a high water requirement would dry up the mix, and aggravate stiffening tendencies. The authors have concluded that the effect of pozzolans will vary depending on the nature of the material but that they will not induce false set in a well-made cement.

EFFECTS ON THE PROPERTIES OF CONCRETE

Very little has been written about the effects of false set on the properties of concrete. One study,[5] reported in 1929, concluded that the effect of false set could be disregarded as far as strength is concerned. These tests were made at a constant water-cement ratio, and compared quickly placed mixes (in which the stiffening was allowed to develop) with mixes which were not placed until after the stiffening had been destroyed by prolonged mixing. Bureau experience has been somewhat the same in the laboratory. However, field experience does not agree with these laboratory findings. Invariably, false setting on the job causes marked reduction in strength even with increased cement content.

It was found that a high water requirement, bleeding and poor workability are always associated with a stiffening cement. If the water-cement ratio is held to the specified limit, additional cement is required. Workability, however, is still impaired. Furthermore, there is a growing suspicion that the unexplainable low durability to freezing and thawing of some concretes was caused, at least in part, by false set of the cement.

On two projects in severe climates, supposedly excellent quality, well-controlled concrete started raveling after only one season's exposure. Fluctuating temperatures resulted in many freezing and thawing cycles. In both cases, the cements had stiffening characteristics. In one case, the principal complaint was that the cement exhibited excessive bleeding. A laboratory study is underway which should shed more light on the effect of false set on durability.

Another effect on concrete is the lack of bond between the matrix and the aggregate or reinforcing. This has been noted on several occasions. The top of a concrete pile (Fig. 2) is an example. This pile was damaged during driving. It will be noted that there is an abundance of clean sockets and an absence of broken aggregate, indicating lack of bond and poor strength in the matrix. At this, as well as at other locations, the concrete cracked excessively during the earlier ages. The piles on this job were brittle during driving. The top portions had to be removed and replaced with concrete caps at considerable extra expense.

Brittleness, excessive cracking, and low strength were also experienced in precast concrete pipe manufactured with a false setting cement. None of these characteristics were evidenced with a normal setting cement which required one bag less cement per cubic yard than the abnormal cement. On jobs where severe stiffening of the concrete has been encountered, it has been found extremely difficult to obtain uniform air entrainment.

Another effect, of more general nature, is the lack of reproducibility between successive batches of concrete made with false-setting cement. Scholer[27] states that it has been necessary to repeat laboratory mixes when the cement originally used was found to have severe stiffening. In both the laboratory and the field, extremely inconsistent results have been found whenever stiffening cements were used.

CHARACTERISTICS OF CALCIUM SULFATE

Unfortunately, the material chosen by the cement industry for set regulation is extremely complicated. Even after 40 years of investigation, the system $CaSO_4$—H_2O still presents a confused picture. Some of this confusion can be attributed to a lack of identification of the state of the particular calcium sulfate under study, which is not easy even when the calcium sulfate is in the pure form. Of course, difficulties in identifying the condition of $CaSO_4$ are greatly magnified when it is interground with portland cement clinker.

Kelley, Southard, and Anderson[28] have accepted six forms of calcium sulfate as being unique and reproducible. They are identified as follows:

Fig. 2—Poor bond between matrix and aggregate on pile top. Damaged area is 7 ft long and a maximum of 3 in. deep.

1. *Gypsum, or calcium sulfate dihydrate.*

2. *α-hemihydrate,* the more stable hemihydrate, prepared by heating at 239 F in an atmosphere of saturated steam. Actually, there is good evidence that α-hemihydrate can contain slightly more or less than the theoretical water content. Autoclave plaster is essentially α-hemihydrate.

3. *β-hemihydrate,* obtained when the dihydrate is partially dehydrated under conditions where a nearly saturated steam atmosphere does not prevail. Stucco plaster is essentially β-hemihydrate. Kettle plasters contain a considerable amount of the β variety, which has markedly different properties from α-hemihydrate.

4. *α-soluble anhydrite,* prepared by heating α-hemihydrate. It may be obtained virtually water-free.

5. *β-soluble anhydrite,* prepared by heating the β-hemihydrate, or by direct heating of gypsum. It always contains some water, usually around 0.2 percent. Most "soluble anhydrite" is essentially the β variety. It rehydrates readily.

6. *Insoluble anhydrite* is prepared by heating either soluble anhydrite or any hydrate to 650 F for 1 hour. The conversion can be made at lower temperatures at slower rates. The optical and thermal properties of natural anhydrite are identical with those of artificially-prepared insoluble anhydrite.

Considerably more information on the properties of these several modifications is given by Kelley, Southard, and Anderson[28] and by Riddell.[29] Of outstanding importance for â false set study are the differences in solubilities and water requirements. For example, Riddell says that the water required to produce a paste of normal consistency with β-hemihydrate is about two and one-half times the quantity required by α-hemihydrate. Like-

wise, Berg and Sveshnikova[30] note that the rate of hydration of β-hemihydrate is two or three times as high as that for α-hemihydrate.

The solubilities are particularly interesting. As far as equilibrium conditions are concerned, gypsum shows only a small change in solubility throughout the temperature range of 50 to 212 F. Below 108 F, it is the least soluble and therefore the stable form of calcium sulfate. Above 108 F, insoluble anhydrite is the least soluble form. Hemihydrate is metastable throughout the temperature range 32-212 F.

Since these equilibrium solubilities may not be expected to prevail during the first few minutes of mixing, the rate of solution is also of importance. This paper is not the proper place for a detailed study of calcium sulfate solubilities, so it is sufficient to note that the saturation solubilities of gypsum and hemihydrate are attained rapidly, while natural anhydrite is dissolved at a somewhat slower rate. A saturated solution of hemihydrate, which would be considerably supersaturated with respect to gypsum, precipitates gypsum within a very few minutes, until the gypsum solubility is approached. Below 108 F, a saturated solution of insoluble anhydrite is supersaturated with respect to gypsum, but is relatively stable nevertheless, and does not precipitate gypsum until after prolonged standing. These solubilities are greatly affected by other soluble salts, such as the alkalies and calcium hydroxide, but this field has not been thoroughly explored for the calcium sulfates other than gypsum.

The location of the transition points between the several calcium sulfates has been argued for many years. Values[31] of 262 F for the loss of 1½ molecules of water from gypsum, and 325 F for the loss of ½ molecule of water from plaster are often given. Differential thermal curves show that these temperatures are valid for conditions of fairly rapid heating in unsealed containers. Posnjak[32] points out that dissociation pressure data cannot be used for this purpose and establishes the transition points from solubility measurements. He shows a transition from gypsum to insoluble anhydrite at 108 F, and a metastable transition from gypsum to hemihydrate at 207 F. These points are confirmed by the thermodynamic calculations of Kelley, Southard, and Anderson.[28] The above temperatures apply only for pure calcium sulfates. Small amounts of organic and inorganic compounds considerably lower transition temperatures.

LABORATORY TESTS FOR FALSE SET

In spite of the many references to false set in the literature, very few tests for its identification have been proposed. One of the early contributors[5] noted that it could be detected in the initial and final sets, which would be quick after 2 minutes mixing, but normal after 5 minutes mixing. Whitworth[10] speaks of a "false initial set," indicating that he detected it with the standard time-of-set apparatus.

The specification test used in 1940 by the Bureau has been briefly described in a previous paragraph. As a result of experience with this test, it was

modified from a fixed water-cement ratio paste to one having a specified ½-minute penetration, so as not to penalize cements of higher water requirement. Also, the penetration was measured 5 minutes rather than 2½ minutes after completion of mixing. The limit on the final penetration was raised from 2 mm to 10 mm, since the previous experiences had shown that more rigid requirements were necessary. The mixing time for the paste is purposely long to overcome slight stiffening tendencies which should not be troublesome in concrete. Within the last year this modified test has been introduced into cement specifications issued by the Bureau. (See Appendix)

The Portland Cement Assn.[33] has used a penetration test, based on the difference in penetration of a ¼-in. needle in the Vicat apparatus in neat cement pastes of approximately normal consistency, determined for two different mixing periods. The pastes were mixed for 3 and 1 minutes, and the penetration determined 15 minutes after mixing. Also at the PCA Laboratory, Lerch used a bleeding test to identify premature stiffening. It was based on the difference in the amount of bleeding obtained on pastes mixed for two different periods. If the additional mixing increased the amount of bleeding, stiffening in the cement was indicated.

Lindsay[34] is using what he terms an "X-test for grab set." This is performed on a trowel-mixed pat having an original consistency similar to normal consistency. The pat is placed in the moist closet and tested frequently with the Gillmore needle. The cement fails if the needle does not penetrate at least ⅛ in. at any time during the first 20 minutes. It appears that this test will respond to either false set or to quick, normal set.

Wilsnack[35] uses a penetration test adapted from the Bureau's test, but differing from it in several respects. He first establishes the water requirement, using the regular 1 cm Vicat needle. However, instead of mixing 3 minutes, he uses three 1-minute mixing periods, separated by two 1½ minute rest periods. The needle is dropped 30 seconds after the last mixing period. The water requirement is that required to produce a penetration of 35 ± 2 mm. Wilsnack believes that the effect of the stiffening tendency on the water requirement can be completely eliminated in this manner. Once the water requirement has been determined, the stiffening test itself is run. For this test he uses a slightly smaller rod, 0.186-in. in diameter, and a 1½ minute mixing period. Penetration readings are taken 3, 4, 5, 7, 10, and 14 minutes after the start of the test. The penetration values are plotted against time, and the area bounded by the curve used to calculate the "percent premature stiffening." By this method, the setting characteristics over 14 minutes are averaged, giving a value claimed to be more quantitative than the Bureau's, although less applicable to specification usage.

One test,[36] not of the penetration type, was offered some years ago. It was developed by an investigator who was concerned with the interference to the normal Vicat setting test when the cement had false setting characteristics. Actually, what he proposed was a new setting test based on the rate of settlement of the cement in a liquid suspension. He proposed to place

15 g of cement in a graduated cylinder, add 40 ml of water, stopper and shake for 1 minute, then to plot the rate of settlement with time. The test was then repeated using white spirits or turpentine instead of water for the suspending medium. Inert materials were found to show little difference between the two liquids. Normal setting cement would show a small difference, while cement having a true, quick set would show a big difference. Since false set would not influence the settling rate, in this test a cement with false set would behave like a normal-setting cement. Therefore, the test could be used to differentiate between true and false early sets. This is similar to Lerch's bleeding test mentioned earlier.

Another test[37] identifies the presence of plaster from solubility studies, because it has been shown that hemihydrate has a considerably greater solubility than does gypsum. Alkalies in solution influence the solubility of calcium sulfate, and the effect of their presence must be recognized and given consideration. Hansen[33] did this by making successive extractions until the concentrations of alkalies in the extracts were inconsequential. Since the liquid extracts from cement pastes during the early stages of hydration are essentially a solution of calcium, sodium, and potassium hydroxides and sulfates, another approach is to establish what the saturation SO_3 concentration of a particular cement extract should be in the presence of gypsum. Actual SO_3 concentrations showing considerable supersaturation can then be attributed to hemihydrate (or soluble anhydrite), which usually correlates with false set in the penetration test.

Differential thermal curves are still another method of identifying hemihydrate in mixtures, but it is difficult to quantitatively evaluate the results on commercial cements.

PROBABLE MECHANISMS OF FALSE SET

Theories on false set attribute most of the false set to dehydrated gypsum, although several mechanisms for the action are offered. In the light of evidence now available, it is interesting to examine these theories on the mechanism by which dehydrated gypsum causes false set.

Easiest to dispose of is the idea that the hydration of hemihydrate to dihydrate dries up the mix. A cement containing 5 percent hemihydrate would require less than 1 percent water (by weight of cement) to convert to gypsum. This is insufficient reduction in water to account for the slump losses which have been encountered.

Brandenburg concluded that the conversion of gypsum to anhydrite by heat resulted in a product which had little solubility, and, therefore, poor retarder efficiency. This would indicate that Brandenburg considered stiffening to be a true, unretarded set. Tests indicate that extracts from false-setting cements are actually greatly supersaturated with respect to gypsum, rather than lacking in available sulfate ion. Poor retarder efficiency is not an explanation.

Forsén's proposal on the respective actions of calcium aluminate trisulfate

and monosulfate is interesting. However, Lerch's work with heat liberation curves showed that plaster of paris does retard set, in contradiction to Forsén's hypothesis.

There is no question that plaster set can occur in pastes, mortars, or concretes. It has been proved by using a pumicite-gypsum combination instead of portland cement, so as to remove any influence of cement hydration. A portion of this pumicite-gypsum mixture was tested as blended. Another portion was heated to convert the gypsum to hemihydrate. Penetration and extract tests on pastes made with these portions compare with cement pastes extracted in an identical manner. A five-sack concrete, made with the heated pumicite-gypsum mixture, showed the same increased water requirement and loss in slump that cement with false set shows.

Although there is no question that plaster set can occur, there is considerable difference of opinion as to whether plaster set explains all cases of stiffening. Bogue's theory on carbonation of alkalies has been examined by means of penetration and extract tests on cement in which false set had been induced by aeration. In this case, it was found that after aeration, when the cement failed the penetration test, the SO_3 content of the extract indicated considerable supersaturation to gypsum. Also, when this aerated cement was heated to 660 F, the stiffening characteristic was removed. A temperature of 660 F does not decompose sodium and potassium carbonates so it is assumed that hemihydrate is converted to insoluble anhydrite.

The action of water is more difficult to explain. The case described by Blank, where false set was attributed to wet clinker, has been encountered by a number of cement chemists. Likewise, it has been noted that venting the grinding mills will relieve some troubles. On the other hand, Hansen and Hunt have suggested that dehydration of gypsum can be minimized by adding small amounts of water to the mills. Possibly this conflict could be satisfactorily resolved if vapor pressures in the mills and silos could be measured and related to the condition of the $CaSO_4$ in the cement.

MEANS OF COMBATING OR PREVENTING FALSE SET

In some cements, false set has been relieved by adding 2 percent gypsum, by weight of cement, at the mixer. In other cases, the addition of 0.2 percent RDA (normally used as a grinding aid for raw mix) is successful. Mechanically, stiffening can sometimes be overcome by prolonged mixing, or by remixing after a short rest period. According to Davis,[38] 0.1 percent mineral oil was used to eliminate early stiffening observed with the portland-pozzolan cement used at Bonneville Spillway Dam. This stiffening, also observed in the same cement without pozzolan, was attributed to fineness.

The overwhelming consensus of those with whom this subject was discussed, as well as the conclusions of the majority of published reports, attributes false set to dehydration of gypsum. As such, it can be corrected in the manufacturing process. Too, conditions of improper composition or nonuniform burning, suggested as contributing to false set, are the manu-

facturer's responsibility. As far as it has been possible to learn, there is not a single reported instance where adequate cooling of the grinding mills and of the cement prior to storage has failed to relieve stiffening.

Improper storage and handling are the only conditions contributing to false set where the consumer shares the responsibility with the producer. Because of this, some manufacturers have felt that they should not be held liable for conditions which develop after the cement leaves the mill. In cases of negligence, no doubt this is true. But it is believed that it is possible for a cement to be stable to a nominal amount of exposure. In fact, some cements have been encountered, including high-alkali ones, in which false set is not induced by aeration over a considerable time.

The use of natural anhydrite, alone or mixed with gypsum, has been proposed on several occasions, as a matter of economy. It has been shown that anhydrite is satisfactory for some cements, but that it does not adequately retard others. Likewise, work with an artificially-prepared anhydrite indicates that satisfactory cements, free from false set, can be prepared with this material. It is possible that the apparently inconsistent action of natural or artificial anhydrite can be explained by considering the action of gypsum on different types of cements. As an example, gypsum does not function as a retarder for low-alumina cements. In fact, the final set of some low alumina clinkers is too slow to meet specifications. Here the gypsum actually accelerates rather than retards, and produces higher strengths.

High C_3A cements are inclined to be quick-setting, and are retarded by gypsum. For high C_3A cements, the anhydrite, having a slower rate of solution, may not be able to furnish sufficient sulfate ion to retard the flash-set tendency. On the other hand, the slower rate of solubility may be sufficient for low C_3A cements. For low C_3A cements, the use of natural or artificial insoluble anhydrite may be a means of avoiding false set.

CONCLUSION

The one conclusion to be drawn from this study is that a solution to the problem of false set is long overdue. The answer might take any one of several forms. One way would be through the use of a mixer addition which would overcome the false-setting characteristics. This "doctoring," however, is not as desirable as a change in manufacturing conditions responsible for false set.

Holding down the grinding temperature through the use of cool clinker and water-sprays on the mills has been very successful. The only objection to this procedure is the housekeeping difficulties introduced.

Using a stable form of calcium sulfate, either natural or artificial anhydrite, is another possibility, particularly for those clinkers that do not require retardation. Perhaps an entirely new set regulator will be found—one that does not have the undesirable characteristics of calcium sulfate.

It is completely unfair to ask a consumer to use a product whose properties are as changeable and unpredictable as a cement with severe false set. Certainly the problem is worthy of the research necessary to provide a solution that will be satisfactory to both consumer and producer.

ACKNOWLEDGMENT

The authors desire to acknowledge the assistance of W. T. Moran, O. J. Glantz and E. T. Oborn in the false set studies conducted in the laboratories of the Bureau of Reclamation.

REFERENCES

1. Meade, Richard K., *Portland Cement*, The Chemical Publishing Co., Easton, Pa., 1926,
2. Eckel, Edwin C., *Cements, Limes, and Plasters*, John Wiley and Sons, Inc., New York. 1928.
3. Meade, Richard K., "Manufacturing Conditions Influencing Setting Time of Portland Cement," *Rock Products*, V. 31, 1928, No. 7, p. 58.
4. Meade, Richard K., "Gypsum and Anhydrite in the Manufacture of Portland Cement," *Rock Products*, V. 31, 1928, No. 24, p. 55; No. 25, p. 40.
5. "The 'False Setting' Time of Cement," *Cement and Cement Manufacture*, V. 2, 1929, p. 14.
6. Brandenburg, H. R., " 'False Set' of Portland Cement," *Cement and Cement Manufacture*, V. 2, 1929, p. 96.
7. C. E. H., " 'False Set' of Cement," *Cement and Cement Manufacture*, V. 2, 1929, p. 158.
8. " 'False Set' of Portland Cement," *Cement and Cement Manufacture*, V. 2, 1929, pp. 230-231.
9. Mehta, D. K., " 'False Set' of Portland Cement," *Cement and Cement Manufacture*, V. 3, 1930, pp. 306-307.
10. Whitworth, Frederick, "False Setting Time of Cement," *Cement and Cement Manufacture*, V. 3, 1930, pp. 573-574.
11. Anzlovar, V. M., "Mill Temperature and the Setting Time of Cements," *Cement and Cement Manufacture*, V. 4, 1931, pp. 381-384.
12. Koyanagi, K., "Action of Gypsum on the Setting of Cement," *Cement and Cement Manufacture*, V. 4, 1931, p. 1258.
13. Whitworth, Frederick, "Further Observations on the False Setting of Cement," *Cement and Cement Manufacture*, V. 4, 1931, pp. 759-762.
14. Whitworth, Frederick, "Effect of Temperature on the Setting Time of Portland Cement," *Cement and Cement Manufacture*, V. 4, 1931, pp. 1001-1002.
15. Schachtschabel, P., "Action of $CaSO_4$ on Ground Clinker," *Cement and Cement Manufacture*, V. 6, 1933, pp. 54-56.
16. Whitworth, Frederick, " 'False Set' of Cement," *Cement and Cement Manufacture*, V. 6, 1933, pp. 179-180.
17. Davis, A. C., *Portland Cement*, Concrete Publications Ltd., London, 1934.
18. Blank, Alton J., "What Conditions Produce False-Setting Cements," *Concrete—Cement Mill Section*, Feb. 1935, pp. 43-46.
19. Lea, F. M. and Desch, C. H., *The Chemistry of Cement and Concrete*, Edward Arnold & Co., London, 1937.
20. Forsén, L., "The Chemistry of Retarders and Accelerators," *Symposium on the Chemistry of Cements*, Stockholm, 1938, pp. 298-394. Including discussion by R. H. Bogue.
21. Lerch, William, "The Influence of Gypsum on the Hydration and Properties of Portland Cement Pastes," ASTM *Proc.* V. 46, 1946, p. 1252.
22. Bogue, R. H., *The Chemistry of Portland Cement*, Reinhold Publishing Corp., New York, 1947.
23. Hansen, W. C. and Hunt, J. O., "The Use of Natural Anhydrite in Portland Cement," ASTM *Bulletin* No. 161, 1949, p. 50.
24. Stadtfeld, Nicolaas T. F., "Inspection and Testing of Materials," ACI JOURNAL, Oct. 1949, *Proc.* V. 46, pp. 237-247.
25. Barona, Fedrico, "Addition of Gypsum to the Distinct Types of Cement that Mexico Produces," *Construccion Moderna*, June 1950.
26. Swayze, M. A., personal communication, Mar. 1950.
27. Scholer, C. H., personal communication, Mar. 1950.

28. Kelley, K. K., Southard, J. C. and Anderson, C. T., "Thermodynamic Properties of Gypsum and Its Dehydration Products," *U. S. Bureau of Mines Technical Paper* 625, 1941, 70 pp.

29. Riddell, W. C., "Physical Properties of Calcined Gypsum," *Rock Products*, V. 53, No. 5, 1950, p. 68.

30. Berg, L. G. and Sveshnikova, V. N., "Modifications of Calcium Sulfate Hemihydrate," *Izvest. Akad. Nauk SSSR*, Otdel, Khim. Nauk, 1946, pp. 19-24. *Chemical Abstracts*, V. 42, p. 6221c.

31. *Handbook of Chemistry and Physics*, 31st Edition, Chemical Rubber Publishing Co., Cleveland, Ohio, 1949, p. 420.

32. Posnjak, E., "The System, $CaSO_4$—H_2O," *American Journal of Science*, V. 5, 1938, No. 35a, p. 247.

33. Hansen, W. C., personal communication, Apr. 1950.

34. Lindsay, G. L., personal communication, Mar. 1950.

35. Wilsnack, G. C., personal communication, Nov. 1950.

36. "Suggestions for a New Setting-Time Test," *Cement and Cement Manufacture*, V. 2, 1929, p. 214.

37. Unpublished data, Bureau of Reclamation, Denver, Colo.

38. Davis, Raymond E., Discussion of a paper by R. R. Clark and H. E. Brown, Jr., 'Portland-Puzzolan Cement as Used in the Bonneville Spillway Dam," ACI JOURNAL, May-June 1937, *Proc.* V. 33, p. 222-1.

APPENDIX

METHOD OF TEST FOR PREMATURE STIFFENING OR "FALSE" SET OF PORTLAND CEMENT

Five hundred grams of cement to be tested shall be kneaded into a paste with the measured quantity of water as described in Federal Specification SS-C-158b (May 1946), Paragraphs F-46 and F-47, except that the mixing time shall be 3 minutes instead of 1½ minutes. The paste shall be quickly formed into a ball with the gloved hands, completing the operation by tossing it six times from one hand to the other with the hands maintained about 6 in. apart. The ball resting in the palm of one hand shall be pressed into the larger end of the conical ring, G (Fig. 3 of SS-C-158b), held in the other hand, completely filling the ring with paste. The excess at the larger end shall then be removed by a single movement of the palm of the hand. The ring shall then be placed on its larger end on a glass plate, the excess paste at the smaller end sliced off at the top of the ring by a single oblique stroke of a sharp-edged trowel held at a slight angle with the top of the ring, and the top smoothed, if necessary, with a few light touches of the pointed end of the trowel. During these operations, care shall be taken not to compress the paste. The paste confined in the ring, resting on the glass plate, shall be centered under the rod, B, the plunger end, C, of which shall be brought in contact with the surface of the paste and the set screw, E, tightened. Then the movable indicator, F, shall be set to the upper zero mark of the scale, and the rod shall be released exactly one-half minute after completion of the initial mixing period. The apparatus shall be free from all vibration during the test. The paste shall be considered to have proper consistency when the rod settles 35 ± 2 mm below the original surface in 30 seconds after being released. Trial pastes shall be made with varying percentages of water until this consistency is obtained.

After completion of the one-half-minute reading, the plunger shall be removed from the paste, cleaned and the ring and plate reset in a new position. This operation should be performed with as little disturbance as possible to the paste confined in the Vicat ring. The plunger shall then again be brought in contact with the surface of the paste, the set screw tightened and the removable indicator set to the upper zero mark of the scale. Five minutes after completion of the initial mixing period the plunger shall be released and the cement shall be considered to show premature stiffening when the needle fails to settle more than 10 mm below the original surface 30 seconds after being released.

Title No. 47-39

A direct procedure for the design of T-beams arrives immediately at an economical section without numerous cut-and-try trials.

Direct Design of T-Beams*

By HENRY J. COWAN†

SYNOPSIS

In designing T-beams by usual methods some initial dimensions of the section must be assumed, either the complete dimensions—and the maximum concrete and steel stresses computed to ensure they are less than the allowable stress—or the effective depth only assumed—with tension area computed and the maximum compressive stress in the flange checked to find if compressive reinforcement is required. The author emphasizes that both methods are likely to produce uneconomical sections and take more time.

The direct design procedure proposed enables the dimensions of a T-beam to be calculated for any given set of conditions. Curves are set up for finding the value of j, and the solution of problems by normal methods and "direct design" are compared by solving six examples. An "equivalent flange" method is proposed for the balanced design of T-beams when the area of compression reinforcement is specified, and results show little difference between exact and approximate methods of design.

INTRODUCTION

T-beams are generally designed by one of two methods. The complete dimensions of the section may be assumed, and the maximum concrete and steel stresses computed to ensure that they are less than the maximum permissible stresses.[1] Alternately the effective depth only may be assumed; the area of tension is then computed, and the maximum compressive stress in the flange checked to see whether compression reinforcement is required.[2]

In both methods the designer must make some initial assumption with regard to the dimensions of the section. The first assumption may be unsuitable, and the dimensions eventually adopted are rarely those which develop the maximum allowable stresses in *both* the steel *and* the concrete. Both methods are therefore apt to be uneconomical in the use of materials and of the designer's time. They are particularly irksome to the student who has insufficient experience to be able to assume reasonable dimensions.

*Received by the Institute July 25, 1950. Title No. 47-39 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 22, No. 7, Mar. 1951, *Proceedings* V. 47. Separate prints are available at 35 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than July 1, 1951. Address 18263 W. McNichols Rd., Detroit 19, Mich.

†Structural Engineer, Lecturer in Civil Engineering, University of Sheffield, England.

A direct design procedure is proposed which enables the dimensions of the section to be calculated for any given set of conditions. In the T-section, as in the rectangular section, the resistance moment is formed by a tensile force provided by the tension reinforcement, and a compressive force provided by the concrete and by the compression reinforcement, if any. Simple expressions can be derived for both these forces. The difficulty lies in finding a suitable formula for the moment arm, jd, in terms of the quantities given. The difficulty is, however, more apparent than real. The rate of variation of the moment arm coefficient, j, is small, and in practice its value can be assessed within 1 percent without difficulty; it can be obtained with a considerable degree of accuracy from the curves shown. It is the author's opinion that a designer will, with a little practice, be able to assign the correct value to j by inspection, without consulting the curves.

The usual assumptions of the elastic working stress theory are made, *viz.*, that both concrete and steel are elastic within the range of the allowable stresses, and that the steel takes all the tensile stresses.[3] Compressive stresses in the concrete rib are ignored.

The flange of the T-beam is normally formed by the floor slab. It is assumed that the dimensions of the slab will have been calculated before the design of the beam is considered, and that the width and thickness of the flange are consequently given.[4]

NOTATION

a'_s = area of additional tension steel in beam with double reinforcement

A_s = area of tension reinforcement

A'_s = area of compression reinforcement

b = width of rectangular beam, width of flange of T-beam

B = "equivalent" width of flange of T-beam with compression reinforcement

C = resultant compressive force in section

d = effective depth of beam

d' = depth of compression reinforcement

ϵ_c = compressive strain at extreme fiber

ϵ_{c1} = compressive strain at center of compression reinforcement (at depth d')

ϵ_{c2} = compressive strain at base of flange (at depth t)

ϵ_t = tensile strain at center of tension reinforcement (at depth d)

E_c = modulus of elasticity of concrete = $1000\,f'_c$

E_s = modulus of elasticity of steel

f_c = actual compressive stress in concrete at extreme fiber

f_{c1} = compressive stress in concrete at depth d'

f_{c2} = compressive stress in concrete at depth t (at base of flange)

f'_c = ultimate compressive strength of concrete

f_s = actual stress in tension reinforcement

f'_s = actual stress in compression reinforcement

F_c = maximum allowable stress for concrete in flexure = $0.45\,f'_c$

F_s = maximum allowable stress for steel

j = moment arm coefficient (ratio of distance between resultant compressive force and resultant tensile force to effective depth)

k = neutral axis depth ratio

M = bending moment

n = modular ratio = $E_s/E_c = 30{,}000/f'_c$

t = thickness of flange of T-beam

T = resultant tensile force in section

BALANCED DESIGN

For any given bending moment there is one section in which both the steel and the concrete are stressed to the full value of the maximum allowable stresses. This is termed a balanced section. For balanced design the stress at the top of the flange, f_c, is equal to the maximum allowable concrete stress F_c, and the stress in the tension reinforcement, f_s, is equal to the maximum allowable steel stress, F_s.

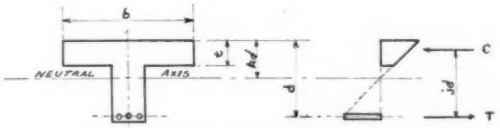


Fig. 1—Design conditions and force diagram for T-beam

On the assumption that plane sections remain plane after bending (Fig. 1)

$$\frac{e_c}{kd} = \frac{e_{c2}}{kd - t} = \frac{e_s}{d - kd}$$

Assuming both steel and concrete to obey Hooke's Law

$$\frac{e_c E_c}{F_c} = \frac{e_{c2} E_c}{f_{c2}} = \frac{e_s E_s}{F_s}$$

Therefore

$$\frac{n F_c}{k} = \frac{n f_{c2}}{k - t/d} = \frac{f_s}{1 - k} \quad \ldots\ldots (1)$$

The neutral axis depth ratio for balanced design (Fig. 2) is therefore the same for T-sections and rectangular sections:

$$k = \frac{n F_c}{n F_c + F_s} \quad \ldots\ldots (2)$$

Ignoring compression in the concrete rib, the resultant compressive force in the section is represented by the area of the trapezium in Fig. 1.

$$C = \frac{F_c + f_{c2}}{2} b\, t \quad \ldots\ldots (3)$$

The resultant tensile force in the section is

$$T = F_s A_s \quad \ldots\ldots (4)$$

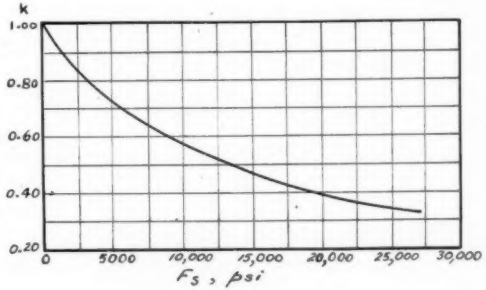


Fig. 2—Neutral axis depth ratio, k, for balanced design. Taking the maximum allowable concrete stress $F_c = 0.45\, f'_c$ and the modular ratio $n = 30{,}000/f'_{c_2}$, $k = nF_c/(nF_c + F_s) = 13{,}500/(13{,}500 + F_s)$. The value of k is therefore independent of the concrete crushing strength.

The moment arm coefficient, j, is obtained by taking moments about the top of the flange:

$$C\,(d - jd) = f_{c2}\,b\,t\,\frac{t}{2} + (F_c - f_{c2})\,\frac{bt}{2} \times \frac{t}{3}$$

Substituting for C from Eq. (3), for f_{c2} from Eq. (1), and dividing by $F_c bt/6kd$

$$3\,d\,(2\,kd - t)\,(1 - j) = 3\,t(kd - t) + t(kd - kd + t)$$

which gives

$$j = 1 - \frac{t(3k - 2t/d)}{3d(2k - t/d)} \qquad (5)$$

Taking moments about the center of the tension reinforcement, the resistance moment of the section is

$$M = C\,jd = F_c\,b\,t\,(1 - t/2kd)\left[1 - \frac{t(3k - 2t/d)}{3d(2k - t/d)}\right]d \qquad (6)$$

This is a quadratic equation for the effective depth, d:

$$d^2 - 2\left[\frac{1 + k}{4k}\,t + \frac{M}{2\,F_c\,b\,t}\right]d + \frac{t^2}{3k} = 0 \qquad (7)$$

Taking moments about the line of action of the resultant compressive force

$$M = T\,j\,d = F_s\,A_s\,j\,d$$

which gives the area of tension reinforcement required:

$$A_s = \frac{M}{F_s\,j\,d} \qquad (8)$$

Example 1

The flange of a T-beam is 4 ft wide by 5 in. thick. Neglecting compressive stresses in the rib, calculate the effective depth and area of reinforcement required for balanced design to resist a bending moment of 3,000,000 in.-lb, taking the ultimate strength of the concrete, $f'_c = 2500$ psi, and the maximum allowable steel stress, $F_s = 18{,}000$ psi.

Maximum allowable concrete stress $F_c = 0.45\,f'_c = 1125$ psi
Modular ratio $n = 30{,}000/f'_c = 12$
Neutral axis depth ratio $k = n\,F_c/(n\,F_c + F_s) = 13{,}500/(13{,}500 + 18{,}000) = 0.429$

$$\frac{1 + k}{4\,k}\,t = \frac{1.429 \times 5}{4 \times 0.429} = 4.16 \text{ in.}$$

$$\frac{M}{2\,F_c\,b\,t} = \frac{3{,}000{,}000}{2 \times 1125 \times 48 \times 5} = 5.55 \text{ in.}$$

$$\frac{t^2}{3\,k} = \frac{5 \times 5}{3 \times 0.429} = 19.4 \text{ sq in.}$$

The effective depth required for balanced design is therefore

$$d = 9.71 + \sqrt{94.2 - 19.4} = 18.35 \text{ in.}$$

From Eq. (5)

$$j = 1 - \frac{5\,(1.287 - 0.544)}{3 \times 18.35\,(0.858 - 0.272)} = 0.883$$

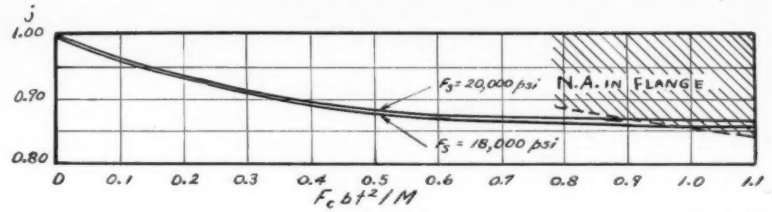


Fig. 3—Moment arm coefficient, j, for the balanced design of T-beams. As $F_c bt^2/M$ increases the depth of the neutral axis decreases. When the neutral axis passes into the flange the section becomes equivalent to a rectangular section, and the value of j then remains constant and equal to $1 - k/3 = (2\,n\,F_c + 3\,F_s)/3\,(n\,F_c + F_s)$.

From Eq. (8) the area of reinforcement required for balanced design is

$A_s = 3{,}000{,}000/18{,}000 \times 0.883 \times 18.35 = 10.28$ sq in.

This procedure is rather lengthy for use in the design office. It is possible to reduce the arithmetical work considerably by taking the value of j from Fig. 3, in which the value of $F_c bt^2/M$ corresponding to any given value of j is obtained by solving the three simultaneous equations (2), (5) and (6).

Eq. (6) may be written in the form

$M = F_c\,b\,t\,(1 - t/2kd)\,jd$, or

$$M/F_c\,b\,t^2 = j\,(d/t - \tfrac{1}{2}k) \qquad (9)$$

Example 2

Solve Example 1 by taking the value of j from Fig. 3.

Neutral axis depth ratio $k = 13{,}500/(13{,}500 + 18{,}000) = 0.429$

$F_c\,b\,t^2/M = 1125 \times 48 \times 25/3{,}000{,}000 = 0.450$

From Fig. 3, $j = 0.883$

From Eq. (9), $1/0.450 = 0.883\,(d/5 - 1/0.858)$

which gives the effective depth for balanced design

$d = 5\,(2.51 + 1.16) = 18.35$ in.

From Eq. (8) the area of reinforcement required for balanced design

$A_s = 3{,}000{,}000/18{,}000 \times 0.883 \times 18.35 = 10.28$ sq in.

UNDER-REINFORCED AND OVER-REINFORCED SECTIONS

If the effective depth of the section is greater than that required for balanced design, the steel attains its maximum allowable stress, F_s, while the maximum stress in the concrete is some value $f_c < F_c$. The steel ratio is then reduced below that required for balanced design, and the section is therefore *under-reinforced.*

It can be shown[5] that the moment arm coefficient for an under-reinforced beam

$$j = 1 - \frac{\dfrac{4\,n\,M}{F_s\,b\,t\,d}\left(1.5 - \dfrac{t}{d}\right) - \dfrac{t}{d}}{\dfrac{6\,n\,M}{F\,b\,t^2}\left(2.0 - \dfrac{t}{d}\right) - \dfrac{t}{d}} \qquad (10)$$

The area of tension reinforcement required for an under-reinforced beam

$$A_s = M/F_s\,j\,d \qquad (11)$$

If the effective depth is less than that required for balanced design, the concrete attains its maximum allowable stress, F_c, while the stress in the steel is some value $f_s < F_s$. The steel ratio must then be increased above that required for balanced design, and the section is therefore *over-reinforced.*

It can be shown[5] that the moment arm coefficient for an over-reinforced beam

$$j = 1 - \frac{\frac{4\,M}{F_c\,b\,d^2} - \frac{t}{d}}{\frac{6\,M}{F_c\,b\,t\,d} - \frac{t}{d}} \qquad (12a)$$

and the maximum steel stress which may be permitted if the concrete is not to be overstressed

$$f_s = n\,F_c \left[\frac{3\left(2\,\frac{d}{t} - 1 - \frac{2\,M}{F_c\,b\,t^2}\right)}{3 - 2t/d} - 1 \right] \qquad (12b)$$

The area of tension reinforcement required for an over-reinforced beam

$$A_s = M/f_s\,j\,d \qquad (13)$$

For curves for the moment arm coefficient for a concrete with a crushing strength of 2500 psi reinforced with structural grade steel see Fig. 4.

The dotted line ABC gives the value of j for balanced design. Above this line the beam is over-reinforced; below it the beam is under-reinforced.

As the bending moment decreases, the depth of the neutral axis decreases also; eventually it passes into the flange. The T-beam then becomes equivalent to a rectangular section. The dotted line ADC gives the values of j for under-reinforced rectangular sections, and the dotted line CE gives the value of j for over-reinforced rectangular sections.

As the bending moment increases, the depth of the neutral axis increases also. If compression in the concrete rib is ignored, this process is rapid, particularly if the ratio t/d is small. The area of the trapezium representing the resultant compressive force in the section can increase only through an increase in the value of f_{c2}. The length of the moment arm cannot increase appreciably without bringing the neutral axis *outside* the section; the area of tension reinforcement required then becomes infinite. This condition is indicated by the shaded portion above the dotted line AF.

For over-reinforced sections a comparatively small increase in the value of the bending moment will therefore result in a large increase in the area of

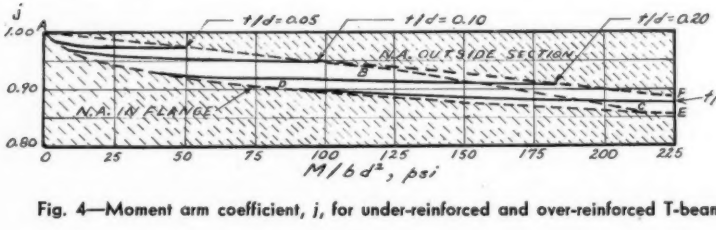


Fig. 4—Moment arm coefficient, j, for under-reinforced and over-reinforced T-beams.

steel required. As a result the problem of over-reinforced design is largely of academic interest.

The range of balanced and under-reinforced sections is contained entirely within the cigar-shaped area $ABCD$. For $M/bd^2 = 207$ psi the only balanced design possible is a rectangular section. For higher values of M/bd^2 the beam is over-reinforced irrespective of the thickness of the flange, and the design is unlikely to be economical.

A study of Fig. 4 shows that the t/d curves are virtually horizontal until the neutral axis approaches the lower face of the flange, when they become asymptotic to the curve for rectangular beams. It is therefore sufficiently accurate to use the value of j appropriate to balanced design until the neutral axis approaches the lower face of the flange. Thereafter the beam may be treated as a rectangular section.

Example 3

Calculate the area of reinforcement required for the beam of Example 1 at sections where the bending moment is (a) 2,500,000 in.-lb, (b) 1,000,000 in.-lb, and (c) 500,000 in.-lb.

$b = 48$ in., $d = 18.35$ in., $t = 5$ in., $F_s = 18{,}000$ psi
$t / d = 5 / 18.35 = 0.272$.

(a) $M / bd^2 = 155$ psi

From Fig. 4 or Fig. 3, $j = 0.89$

The area of tension reinforcement required is therefore

$A_s = 2{,}500{,}000 / 18{,}000 \times 0.89 \times 18.35 = 8.50$ sq in.

(b) $M/bd^2 = 62$ psi

From Fig. 4 or Fig. 8, $j = 0.91$
$A_s = 1{,}000{,}000 / 18{,}000 \times 0.91 \times 18.35 = 3.33$ sq. in

(c) $M/bd^2 = 31$ psi (The neutral axis is inside the flange, and the beam is therefore equivalent to a rectangular section.)

From Fig. 4 or Fig. 8, $j = 0.94$
$A_s = 500{,}000 / 18{,}000 \times 0.94 \times 18.35 = 1.61$ sq in.

BALANCED DESIGN OF T-BEAMS WITH COMPRESSION REINFORCEMENT

If the effective depth is specified

It has been shown that over-reinforced T-sections are likely to be uneconomical. If the effective depth is limited to less than that required for balanced design, it is usually necessary (and always advisable) to introduce a sufficient quantity of compression reinforcement to balance the design.

The resistance moment of the section may be treated as two parts: a moment formed by the concrete flange with a balanced area of tension reinforcement, and a moment formed by the compression reinforcement with the remainder of the tension reinforcement (Fig. 5).

For balanced design (from Eq. (6))

$$M_1 = F_c\, b\, t\, (1 - t/2kd)\, jd \qquad (14)$$

where j is obtained from Fig. 3.

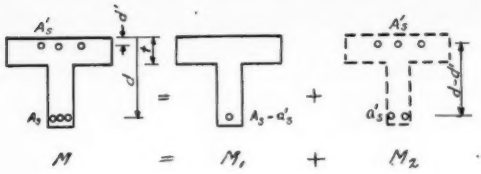


Fig. 5—Resistance moment of section is summation of that formed by flange with balanced tension reinforcement and that formed by compression reinforcement with remainder of tension reinforcement.

The area of tension reinforcement required to balance the compressive force in the concrete flange

$$A_s - a'_s = M_1/F_s\, j\, d \quad (15)$$

The remainder of the resistance moment is provided by additional tension and compression reinforcement.

$$M_2 = M - M_1 \quad (16)$$

The resistance moment of the additional steel is

$$M_2 = F_s\, a'_s\, (d - d') = f'_s\, A'_s\, (d - \mathrm{d}') \quad (17)$$

and therefore

$$a'_s = M_2 / F_s\, (d - d') \quad (18)$$

In balanced design the tension steel is stressed to its maximum allowable stress F_s. According to the elastic theory, the stress in the compression steel (Fig. 6)

$$f'_s = \frac{kd - d'}{d - kd} F_s \quad (19a)$$

If the effectiveness of the compression reinforcement is doubled, in accordance with Section 706 (*b*) of the ACI Code,

$$f'_s = 2\,\frac{kd - d'}{d - kd} F_s \quad (19b)$$

From Eq. (17) and (19), the area of compression reinforcement required is therefore

$$A'_s = a'_s \frac{d - kd}{kd - d'} \quad (20a)$$

or, if the design is in accordance with Sec. 706 (*b*),

$$A'_s = \tfrac{1}{2}\, a'_s \frac{d - kd}{kd - d'} \quad (20b)$$

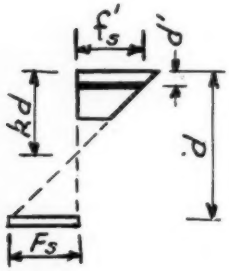


Fig. 6—Stress diagram for compression steel

The area of tension reinforcement required is obtained from Eq. (15) and (18):

$$A_s = (A_s - a'_s) + a'_s \quad (21)$$

Example 4

Using compression steel, calculate the area of reinforcement required for the beam of Example 1 at a section where the bending moment is 5,000,000 in.-lb

$b = 48$ in.; $d = 18.35$ in.; $t = 5$ in.; $d' = 2.25$ in. (assumed);
$F_s = 18{,}000$ psi; $F_c = 1125$ psi; $n = 12$

The initial calculations are identical with those of Examples 1 or 2,

i.e., $k = 0.429$; $j = 0.883$; $M_1 = 3{,}000{,}000$ in.-lb.
$A_s - a'_s = 10.28$ sq in.
$M_2 = 5{,}000{,}000 - 3{,}000{,}000 = 2{,}000{,}000$ in.-lb

From Eq. (18)

$$a'_s = 2{,}000{,}000 / 18{,}000\ (18.35 - 2.25) = 6.90 \text{ sq in.}$$

From Eq. (21)

$$A_s = 10.28 + 6.90 = 17.18 \text{ sq in.}$$

From Eq. (20), using the elastic theory,

$$A'_s = 6.90 \times \frac{18.35 - (0.429 \times 18.35)}{(0.429 \times 18.35) - 2.25} = 12.90 \text{ sq in.}$$

If the design is in accordance with Sec. 706 (*c*) of the ACI Code

$$A'_s = 12.90 / 2 = 6.45 \text{ sq in.}$$

If the area of compression reinforcement is specified

The area of compression reinforcement may sometimes be specified, instead of the effective depth. This is likely to be the case if (a) some of the reinforcement from a neighboring section is carried through to obtain a simpler arrangement of the reinforcement; (b) if due to eccentricity of the load there is a torsional moment which requires reinforcement on both faces; or (c) if due to a different disposition of the live load there is a reversal in the sign of the bending moment which requires reinforcement on both faces of the section.

If the design is to be balanced, $f_c = F_c$, $f_s = F_s$, and $k = nF_c/(nF_c + F_s)$.

The total compressive force in the section (Fig. 7) is

$$C = \tfrac{1}{2}\, b\, t\, (F_c + f_{c2}) + A'_s f'_s \quad (22)$$

Substituting $f_{c2} = \dfrac{F_c\,(kd - t)}{kd}$ (23)

and $f'_s = \dfrac{n\,F_c\,(kd - d')}{kd}$ (24)

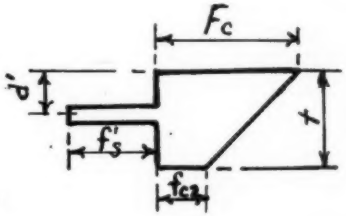


Fig. 7—Composite stress diagram for T-beam flange

$$C = \frac{F_c\,[b\,t\,(2\,kd - t) + 2\,n\,A'_s\,(kd - d')]}{2\,kd} \quad \ldots\ldots (25)$$

Taking moments about the upper face of the flange (Fig. 7)

$$C\,(d - jd) = f_{c2}\,b\,t\,\frac{t}{2} + \frac{f_c - f_{c2}}{2}\,b\,t\,\frac{t}{3} + n\,A'_s f'_s\,d'$$

Substituting for C, f_{c2}, and f_s' from Eq. (25), (23) and (24):

$$\frac{F_c}{2\,kd}\,[b\,t\,(2\,kd - t) + 2\,n\,A'_s\,(kd - d')]\quad (1 - j)\,d =$$

$$\frac{F_c}{2\,kd}\left[\frac{b\,t^2}{3}\,(3\,kd - 2t) + 2\,n\,A'_s\,(kd - d')\,d'\right]$$

which gives the moment arm coefficient

$$j = 1 - \frac{b\,t^2\,(3\,kd - 2t) + 6\,n\,A'_s\,(kd - d')\,d'}{3\,d\,[b\,t\,(2kd - t) + 2\,n\,A'_s\,(kd - d')]} \quad \ldots\ldots (26)$$

The resistance moment of the section

$$M = C\,jd$$

$$= \frac{F_c}{2\,kd}\,[b\,d\,(2kd - t) + 2\,n\,A'_s\,(kd - d')]$$

$$\left[1 - \frac{b\,t^2\,(3kd - 2t) + 6\,n\,A'_s\,(kd - d')\,d'}{3\,d\,[b\,t\,(2kd - t) + 2\,n\,A'_s\,(kd - d')]}\right] d$$

Dividing by $F_c\,b\,t\,/\,d$

$$\frac{Md}{F_c\,b\,t} = \left(1 - \frac{t}{2kd}\right) d^2 + \frac{n\,A'_s}{b\,t}\left(1 - \frac{d'}{kd}\right) d^2 - \left(\tfrac{1}{2} - \frac{t}{3kd}\right) td - \frac{n\,A'_s}{b\,t}\left(1 - \frac{d'}{kd}\right) d'\,d$$

This is a quadratic equation for the effective depth, d:

$$d^2\left[1 + \frac{n\,A'_s}{b\,t}\right] - 2\,d\left[\frac{1 + k}{4k}\left(t + 2\,\frac{n\,A'_s}{b\,t}\,d'\right) + \frac{M}{2\,F_c\,b\,t}\right] +$$

$$\left[\frac{t^2}{3\,k} + \frac{n\,A'_s}{b\,t} \times \frac{d'^2}{k}\right] = 0 \quad \ldots\ldots (27)$$

Having calculated the effective depth, the area of tension reinforcement required is obtained from

$$A_s = M\,/\,F_s\,j\,d \quad \ldots\ldots (28)$$

Example 5

The flange of a T-beam is 4 ft wide by 5 in. thick and is reinforced with 12.90 sq in. of compression steel at a depth of 2.25 in. Calculate the effective depth and the area of tension reinforcement required for balanced design to resist a bending moment of 5,000,000 in.-lb taking the ultimate strength of the concrete, f_c' as 2500 psi and the maximum allowable steel stress, F_s, as 18,000 psi.

$F_c = 0.45\,f'_c = 1125$ psi; $n = 30{,}000\,/\,f'_c = 12$
$k = n\,F_c\,/(n\,F_c + F_s) = 0.429$
$b = 48$ in.; $t = 5$ in.; $A'_s = 12.90$ sq in.; $d' = 2.25$ in.
$n\,A'_s\,/\,b\,t = 0.645$; $t + 2\,n\,A'_s\,d'\,/\,b\,t = 7.903$ in.
$(1 + k)/\,4\,k = 0.833$; $M\,/\,2\,F_s\,b\,t = 9.25$ in.

$t^2\,/\,3\,k = 19.42$ sq in.; $\dfrac{n\,A'_s\,d'^2}{b\,t\,k} = 7.60$ sq in.

Substituting these values into Eq. (27)

$$1.645\,d^2 + 2\,d\,[(0.833 \times 7.903) + 9.25] + (19.42 + 7.60) = 0$$

This gives the effective depth

$$d = 9.62 + \sqrt{92.7 - 16.4} = 18.35 \text{ in.}$$

The moment arm coefficient is obtained from Eq. (26)

$$j = 1 - \frac{48 \times 5^2\,(24.6 - 10.0) + [6 \times 12 \times 12.9(7.87 - 2.25)] \times 2.25}{3 \times 18.35\,[48 \times 5(15.8 - 5.0) + 2 \times 12 \times 12.9(7.87 - 2.25)]} = 0.877$$

From Eq. (28)

$$A_s = 5{,}000{,}000 \,/\, 18{,}000 \times 0.877 \times 18.35 = 17.18 \text{ sq in.}$$

This procedure is clearly too complicated for normal practical design. In the majority of problems, however, a close approximation can be obtained by using an *equivalent width of flange*. If $d' < t \,/\, 3$ this approximation is always on the safe side.

For the purpose of computing the equivalent width of flange, B, the depth of the center of gravity of the compression reinforcement is assumed to be equal to half the thickness of the flange,

i.e., $d' = t \,/\, 2$.

The stress in the compression reinforcement is then

$$f'_s = \tfrac{1}{2}\,n\,(F_c + f_{c2})$$

The total compressive force in the section, from Eq. (22) is

$$C = \tfrac{1}{2}\,b\,t\,(F_c + f_{c2}) + A'_s f'_s$$
$$= \tfrac{1}{2}\,(b\,t + n\,A'_s)\,(F_c + f_{c2})$$

Putting $B = b + n\,A'_s \,/\, t$

$$C = B\,t\,(F_c + f_{c2}) \,/\, 2$$
$$= F_c\,B\,t\,(2\,kd - t) \,/\, 2\,kd \qquad (29)$$

The concrete flange of width b together with the compression reinforcement of cross-sectional area A_s' can therefore be replaced by a plain concrete flange of width B. The section may then be designed by using the procedure for T-beams with tension reinforcement only.

Example 6

Repeat Example 5, using the "equivalent flange" method

$F_s = 18{,}000$ psi; $F_c = 1125$ psi; $n = 12$; $k = 0.429$;
$b = 48$ in.; $t = 5$ in.; $A'_s = 12.90$ sq in.; $M = 5{,}000{,}000$ in.-lb

From Eq. (29) the width of the equivalent flange

$B = 48 + 12 \times 12.90 \,/\, 5 = 78.96$ in.

$F_c\,B\,t^2 \,/\, M = 0.444$ From Fig. 2, $j = 0.88$

From Eq. (9)

$$1/0.444 = 0.88\,(d \,/\, 5 - 1 \,/\, 0.858)$$

which gives the effective depth required for balanced design

$$d = 5\,(2.56 + 1.16) = 18.6 \text{ in.}$$

From Eq. (8) the area of tension reinforcement required

$$A_s = 5{,}000{,}000/18{,}000 \times 0.88 \times 18.6 = 17.0 \text{ sq in.}$$

A comparison with the result obtained in Example 5 shows that there is little difference between the dimensions calculated by the exact and the approximate "equivalent flange" methods.

SUMMARY

1. In beams with tension reinforcement only, given the width and thickness of the flange, the maximum allowable stresses for the steel and the concrete, the modular ratio, and the bending moment at the section, the effective depth, $d_{balanced}$, required for *balanced design* is obtained from the equation

$$M/F_c bt^2 = j\,(d_{balanced}/\,t - 1\,/\,2k) \quad (9)$$

The neutral axis depth ratio for balanced design

$$k = n\,F_c\,/\,(n\,F_c + F_s) \quad (2)$$

2. If the effective depth is increased beyond $d_{balanced}$, the section becomes *under-reinforced.*

3. The area of tension reinforcement required for balanced and under-reinforced sections

$$A_s = M\,/\,F_s jd \quad (8)$$

4. If the effective depth is reduced below $d_{balanced}$, the section becomes *over-reinforced.* The range within which over-reinforced sections are theoretically possible is small, and the design is likely to be uneconomical. *Compression reinforcement* should therefore be used where the depth of the section is limited.

5. In designing the tension and compression reinforcement of a T-section whose over-all dimensions are given, the resistance moment of the section may be treated as made up of two parts: a moment formed by the concrete flange with a balanced area of tension reinforcement, and a moment formed by the compression reinforcement with the remainder of the tension reinforcement (See Eq. (14) to (21).

6. The presence of a given area of compression reinforcement may be allowed for by increasing the width of the flange from b to

$$B = b + n\,A'_s\,/\,t \quad (29)$$

The effective depth and the area of tension reinforcement required are then obtained from Eq. (9) and (8) respectively.

7. The rate of variation of the *moment arm coefficient* for T-beams is small within the working range of normal design. The value of the moment arm coefficient may be obtained from Fig. 3 and 4.

REFERENCES

1. *Reinforced Concrete Design Handbook*, American Concrete Institute, Detroit, 1948, Example 11.

2. *Ibid*, Example 3.

3. "Building Code Requirements for Reinforced Concrete (ACI 318-47)", American Concrete Institute, Detroit, 1947, Section 601.

4. *Ibid*, Section 705 (*a*) and (*b*).

5. Cowan, H. J., "The Direct Design of Reinforced Concrete T-Beams," Part II, *Civil Engineering and Public Works Review*, London, V. 45, 1950, p. 38.

Discussion of a paper by Henry J. Cowan:

Direct Design of T-Beams*

By L. S. MULLER, ROBERT V. HAUER and AUTHOR

By L. S. MULLER†

The writer would like to draw the attention to the fact that the solution of the problem was given a long time ago by Professor Saliger (*Der Eisenbeton*, A. Kröner, Leipzig, 1925, p. 238). He arrives to the same quadratic equation (7) as the author but he says immediately that its use is somewhat cumbersome, and therefore he offers a more elegant solution, the main steps of which may be given as follows:

Using the notation of the author and $\psi = t/d$

From Eq 5: $j = 1 - \dfrac{t(3k - 2t/d)}{3d\ (2k - t/d)} = \dfrac{6k - 3\psi - 3k\psi + 2\psi^2}{3(2k - \psi)}$

Also, as $T = C$

$$A_s F_s = \frac{bt\ F_c\ (2kd - t)}{2kd} \qquad A_s = \frac{\psi\ (2k - \psi)}{2k} \cdot \frac{F_c}{F_s} \cdot bd$$

From Eq. 2: $\dfrac{F_s}{F_c} = \dfrac{1 - k}{k} \cdot n$

Therefore

$$A_s = \frac{\psi\ (2k - \psi)}{2k} \cdot \frac{k}{(1 - k)n} \cdot bd = \frac{\psi\ (2k - \psi)}{2n\ (1 - k)} \cdot bd = \mu\, bd$$

Finally: $M = jd\ C = \dfrac{(6k - 3\psi - 3k\psi + 2\psi^2)d}{3\ (2k - \psi)} \cdot \dfrac{bd\ \psi\ (2k - \psi)\ F_c}{2k} =$

$$M = \frac{[6k - 3\psi\ (1 + k) + 2\psi^2]\ \psi\ F_c}{6k} \cdot bd^2$$

and

$$d = \sqrt{\frac{6k}{[6k - 3\psi\ (1 + k) + 2\psi^2]\ \psi\ F_c}} \cdot \sqrt{\frac{M}{b}} = r\sqrt{\frac{M}{b}}$$

As $\psi = \dfrac{t}{d} = \dfrac{t}{r\sqrt{M/b}}$

$$\frac{1}{t}\sqrt{M/b} = \frac{1}{r\psi} = \sqrt{\frac{[6k - 3\psi\ (1 + k) + 2\psi^2]\ F_c}{6k\psi}}$$

To given F_s and F_c and to different values of ψ the values of r, μ and $\dfrac{1}{t}\sqrt{M/b}$ have been tabulated. By the index value $\dfrac{1}{t}\sqrt{M/b}$ the factor r is

*ACI JOURNAL, Mar. 1951, *Proc.* V. 47, p. 533. Disc. 47-39 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 23, No. 4, Dec. 1951, *Proceedings* V. 47.
†Engineer, Palestine Land Development Co., Jerusalem, Israel.

found from the table, and in the *same* slide-rule position multiplied by $\sqrt{M/b}$ to determine d. Also with μ in the same row $bd\ \mu$ gives A_s.

As the Editor of *Concrete and Constructional Engineering* once rightly pointed out in connection with the work of René Feret: it is a melancholic experience that the same scientific truth or solution must be rediscovered from time to time again and again, as it will be forgotten several times, causing unnecessary waste of time and energy of research workers.

The above example proves once again how important it would be to compile for structural engineering a systematic international bibliographical index, as it has been done in chemistry, which would inform on author, publication, and time of all the successful solutions of problems.

By ROBERT V. HAUER*

The formulas derived in Mr. Cowan's paper are aimed at the design of a so-called "balanced" T-section, where the maximum allowable stresses in both the steel and the concrete are utilized to the limit. However, contrary to the author's opinion expressed in the introduction, the writer believes that such a design is far from being economical. Example No. 1, for instance, arrives at a steel area of more than 10 sq in. in a web with an effective depth of only 18.35 in. Under ordinary conditions, such a shallow section is not only uneconomical on account of the great amount of steel required, but also undesirable because of the necessarily large deformations, which may promote excessive cracks. The disproportion between the amount of reinforcing and the depth of the section is even more pronounced in Example No. 4.

It is evident that the economy of an individual section is only of secondary interest, because, in the first place, a good design should be concerned with the economy of the structure as a whole. But even if this broader aspect of economy is disregarded and the discussion confined to the T-section alone, it must be emphasized that for such a section the most favorable solution is practically independent of the allowable concrete stress. This is readily explained by the fact that the concrete stress is very much influenced by the width of the flange, but that, on the other hand, the flange is a structural part established before the design of the beam and, therefore, unable to affect the economy of the rest of the T-section.

According to the writer's experience, a good guide for estimating reasonable T-beam dimensions are the formulas

$$d = \sqrt[3]{100M} \text{ and } b' = 1/2 \text{ to } 2/3\ d$$

where d and b' are the effective depth and the width of the web respectively, both in inches, and M the bending moment in kip-feet.

These dimensions will keep the concrete stress of the T-beam within the allowable limit and, for a steel stress of 20,000 psi, lead to a steel area of 1.0 to 1.4 percent of $b'd$, which, in most cases, is close to the economical optimum.

If the stresses in the web are neglected, the resultant of the compressive stresses is located somewhat above the center of the flange thickness t. It

*Structural Engineer, Albert Kahn, Inc., Detroit, Mich.

follows that, with sufficient accuracy for all practical purposes, the internal moment arm can be assumed to be $d-0.4\ t$, from which the required tensile reinforcement is easily calculated.

AUTHOR'S CLOSURE

The author is much indebted to Mr. Muller for drawing attention to Professor Saliger's earlier solution of the problem. There is, in point of fact, a still older solution, which is given in the *Second Report of the Joint Committee on Reinforced Concrete*, convened by the Royal Institute of British Architects, London, in 1911, and is still to be found in some English textbooks on architectural building construction.

In both these methods the equations for the moment are very lengthy, because the algebraic expression in Eq. 5 is substituted for the moment arm coefficient j. If this is avoided, as in Eq. 8, by using the numerical value of j in place of the algebraic expression, the computation becomes quite simple, as demonstrated in Example 2. This abridged procedure is made possible by the fact that the moment arm coefficient is virtually constant over the working range, as shown in Fig. 3. This makes it possible to dispense with the design chart without serious loss of accuracy.

Mr. Hauer's contribution is valuable because it draws attention to a point which has not been sufficiently stressed by the author. He rightly points out that in normal building construction the under-reinforced design represents the economical solution. This case is covered in the paper in Example 3. Balanced design becomes economical for long-span continuous girder bridges, and for many strip foundations of the inverted T-beam type, particularly when a high foundation pressure is permissible.

ADDENDUM

Note on the construction of Fig. 4

Lower limiting line *ADCE*—When the neutral axis passes into the flange, the T-beam becomes equivalent to a rectangular section. The variation of the moment arm coefficient for a rectangular section with $M\ /\ b\ d^2$ is shown by the dotted line *ADCE*.

For $M\ /\ b\ d^2 < \frac{1}{2}\ F_c\ b\ kd$ the section is *under-reinforced* (Line *ADC*).

The resistance moment of an under-reinforced section

$$M = \tfrac{1}{2}\ f_c\ b\ kd\ jd$$

where $f_c < F_c$

Substituting $k = 3\ (1 - j)$ and $f_c = \dfrac{F_s}{n}\ \dfrac{k}{1 - k}$

$$M = \tfrac{1}{2}\ \frac{F_s}{n}\ \frac{3\ (1 - j)}{(3\ j - 2)}\ 3\ (1 - j)\ j\ b\ d^2$$

which is a cubic equation for j (Fig. A):

$$j^3 - 2\ j^2 + j\left(1 - \frac{2\ n\ M}{3\ F_s\ b\ d^2}\right) + \frac{4}{9}\ \frac{n\ M}{F_s\ b\ d^2} = 0 \qquad \text{.....(30)}$$

For $M\ /\ b\ d^2 = \frac{1}{2}\ F_c\ b\ d^2$ the design is *balanced* (Point *C*).

For $M\ /\ b\ d^2 > \frac{1}{2}\ F_c\ b\ d^2$ the section is *over-reinforced* (Line *CE*).

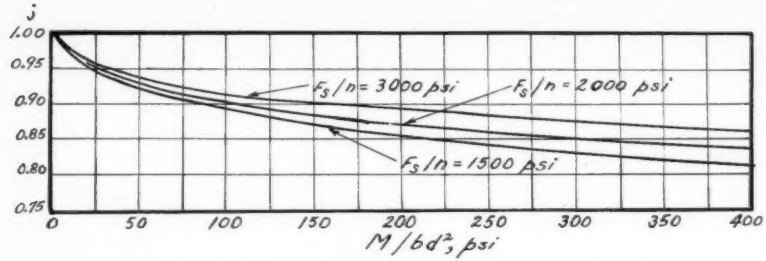


Fig. A—Moment arm coefficient for under-reinforced rectangular sections

The resistance moment of an over-reinforced section
$M = \frac{1}{2} F_c\, b\, kd\, jd = \frac{3}{2} F_c\, b\, (1 - j)\, d \times jd$
This is a quadratic equation for j (Fig. B):

$$j = \frac{1}{2} + \sqrt{\frac{1}{4} - \frac{2M}{3 F_c\, b\, d^2}} \qquad (31)$$

Upper limiting line AF—As the neutral axis approaches the center of gravity of the tension reinforcement, the stress which may be permitted in the steel without overstressing the concrete at the top of the flange approaches zero, and the area of reinforcement required therefore approaches infinity.

Substituting $f_s = 0$ in Eq. (12b)

$$3\left(2\frac{d}{t} - 1 - \frac{2M}{F_c\, b\, t^2}\right) = 3 - 2\frac{t}{d}$$

Multiplying by $F_c\, t^2 / 2\, d^2$

$$\frac{M}{b\, d^2} = F_c\left[\frac{t}{d} - \left(\frac{t}{d}\right)^2 + \frac{1}{3}\left(\frac{t}{d}\right)^3\right] \qquad (32)$$

The dotted line AF is obtained by solving the two simultaneous equations (12) and (32).

Intermediate lines—The dotted line ABC gives the values of j for balanced design. It is obtained from Fig. 3 by multiplying the reciprocal of $F_c\, b\, t^2 / M$ by $F_c\, (d/t)^2$.

Region $ABCD$ is the range of under-reinforced sections, for which the value of j is given by Eq. (10).

Region $ABCF$ is the range of over-reinforced sections, for which j is given by Eq. (12a).

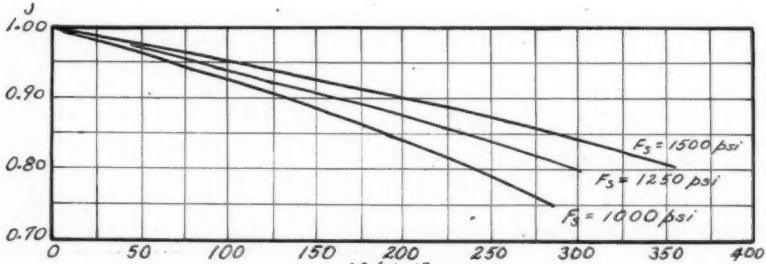


Fig. B—Moment arm coefficient for over-reinforced rectangular sections

Title No. 47-40

Where blast furnace slag is available, cement made of the slag blended with portland clinker is useful in controlling alkali-aggregate expansion.

Alkali-Aggregate Expansion Corrected with Portland-Slag Cement*

By FEDERICO BARONA DE LA O†

SYNOPSIS

Granulated basic blast furnace slag meeting ASTM Spec. C205 should not be considered a pozzolan, but a latent cement with a high siliceous glass content requiring the presence of hydrated lime and gypsum for hydration. To correct alkali-aggregate expansion, a high proportion of slag (50 to 60 percent) should be used, which does not reduce strength, rather than lower percentages (20 to 30) which can not be exceeded with pozzolans unless strength is sacrificed. Corrective action was investigated using Pyrex glass as reactive aggregate, and *NaOH* to increase the alkali content of the different cements and blends to the same high value (1.23 percent).

Under similar conditions, with the same high alkali content, much smaller expansions were obtained with slag blends than with straight portlands. It is not intended to compare or recommend the use of slag instead of pozzolans, or the use of portland-blast furnace slag cement instead of modified, low-heat, or sulfate resistant cement, since in each case, the availability, cost and special conditions will determine what to use. In many cases, slag can be used advantageously to replace 50 to 60 percent of portland clinker.

INTRODUCTION

Slag with a high lime content (over 40 percent), properly prepared by rapid cooling to obtain supercooled glasses of calcium silicates and aluminates (granulated basic slag),[1] and ground together with gypsum and hydrated lime or portland clinker (considered as activators or catalyzers by Lea[2] and set retarders by Forsén[3]), produces hydraulic cement with satisfactory strength, durability and resistance to chemical attack.

As pointed out by Lea[2] the activity of the slag not only depends on its composition but also on its rapid cooling, so as to inhibit crystallization, secure a high amount of glass and prevent the formation of crystalline inactive compounds. It is better to quench the molten slag with water than

*Received by the Institute Dec. 6, 1950. Title No. 47-40 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 22, No. 7, Mar. 1951, *Proceedings* V. 47. Separate prints are available at 35 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than July 1, 1951. Address 18263 W. McNichols Road, Detroit 19, Mich.

†Member American Concrete Institute, Head, Technical Department of Materials, Hydraulic Resources Ministry, Mexico.

in cool air; some of the sulfur (5 to 10 percent) is thus eliminated as H_2S gas, or as CaS dissolved in the water.

Forsén[3] explained that if no set retarders are present, early hydration of the aluminates coats the slag particles and prevents the slow but efficient production of calcium silicate gel that accounts for the slag hydraulicity. That is, no proper hydration is secured unless the early formation of alumina gels is prevented. Therefore, an adequate supply of lime, gypsum or some other retarder is necessary to develop the slag activity.

Although hydraulic cement (slag cement, *ciment de laitier* or *Schlackenzement*) can be made with no portland clinker, by grinding granulated basic slag with approximately 15 percent hydrated lime and gypsum (about 2 percent SO_3), it is too slow for many purposes. It is generally preferred to grind 35 to 65 percent portland clinker with 65 to 35 percent slag and enough gypsum to provide 2 to 3 percent SO_3. Such hydraulic cement is being used in many countries. In Europe, its higher resistance to chemical attack (sea and sulfate waters) has favored its use for sea work[2] and its low heat of hydration for mass concrete construction.[4]

PROPERTIES OF SLAG, CLINKER AND PORTLAND-BLAST FURNACE SLAG CEMENT

Composition

Table 1 shows the composition of the granulated slag[5] produced in Monterrey, Mexico. Three typical analyses show that the composition is quite uniform and that it complies with ASTM specifications. The alkali content is low. In the 1949 samples the lime content slightly exceeds 50 percent.

Table 2 shows the composition of the portland clinker (ASTM Type II) used to produce the blended cement. The clinker complies with Mexican requirements for efficient hydraulic work.[6] For general work the slag-clinker ratio is 60/40 plus an adequate amount of gypsum (2 to 3 percent SO_3). Special grindings for hydraulic work have been made with a 50/50 slag-clinker ratio. The first combination is designated cement "60," the second cement "50." Typical analyses of both cements are shown in Table 2. The alkali content is low in the clinker, lower in the slag, and about midway in both blended cements.

TABLE 1—CHEMICAL COMPOSITION OF MONTERREY BLAST FURNACE SLAG

Chemical constituent, percent	Monterrey blast-furnace slag			Recommended values, ASTM C 205-48T
		1949		
	1944	No. 464	No. 495	
SiO_2	32.8	31.1	30.3	30 to 40
Al_2O_3	15.3	14.2	15.1	8 to 18
FeO	0.8	1.3	1.3	0 to 1
CaO	47.5	50.6	50.3	40 to 50
Mn_2O_3	0.7	—	—	0 to 2
MgO	0.8	1.8	1.4	0 to 8
S (sulfide)	1.3	1.6	1.4	0 to 2
Loss on ignition	—	0.5	0.8	
$\frac{CaO + MgO + \frac{1}{3} Al_2O_3}{SiO_2 + \frac{2}{3} Al_2O_3}$	1.24	1.41	1.41	Not less than 1
Alkalies				
Na_2O	—	0.07	0.07	
K_2O	—	0.13	0.08	
$Na_2O + 0.658\ K_2O$	—	0.15	0.12	

The SO_3 limit in ASTM Specification C205-48T (not higher than 2.5 percent) is considered too low. By going up to about 3 percent, higher strengths are obtained in the Mexican portland-blast furnace slag cement

TABLE 2—COMPOSITION OF CEMENTS

Constituent, percent	Monterrey clinker, Type II	Cements made with Monterrey clinker, Type II			Allowable limits		
		Portland	Portland-slag		US Federal Spec. SS-C-192 (Portland)		ASTM C 205-48T (Portland-slag)
			60	50	Type II	Type IV	
SiO_2	24.3	23.2	26.3	24.8	Min 21.0	Min 22.0	
Al_2O_3	3.8	3.7	10.0	9.5	Max 6.0	Max 6.0	
Fe_2O_3	3.3	3.5	2.5	1.9	Max 6.0	Max 6.5	
CaO (Combined)	64.8	62.6	54.3	56.3			
CaO (Free)	0.6	1.4	0.6	0.4			
Mn_2O_3				0.3			Max 1.5
MgO	1.4	1.3	1.5	1.4	Max 5.0	Max 5.0	Max 5.0
SO_3	0.1	2.0	2.1	3.2	Max 2.0	Max 2.25	Max 2.5
S (Sulfide)	—	—	0.8	0.7			Max 2.0
Insoluble residue	0.6	0.2	0.9	0.8	Max 0.75	Max 0.75	Max 1.0
Loss on ignition	0.5	1.4	1.1	1.4	Max 3.0	Max 2.5	Max 3.0
Compounds							
C_3S	48.7	42.9	—	—	Max 50.0	Max 35.0	
C_2S	33.0	34.2	—	—	—	Min 40.0	
C_3A	4.5	3.9	—	—	Max 8.0	Max 7.0	
C_4AF	10.0	10.6	—	—			
Alkalies							
Na_2O	0.24	0.27	0.16	0.16			
K_2O	0.31	0.38	0.22	0.21			
$Na_2O + 0.658\ K_2O$	0.44	0.52	0.30	0.30	Max 0.60	Max 0.60	
Heat of hydration			cal per g				
3 days			55	68			
7 days			64	77	Max 70	Max 60	
28 days			68	85	Max 80	Max 70	

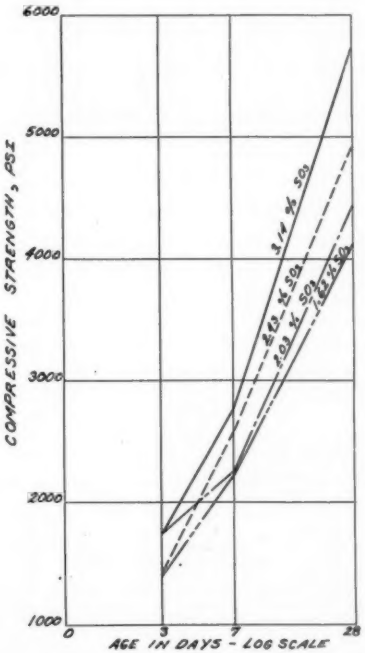


Fig. 1—Compressive strength of portland-blast furnace slag cement in relation to SO_3 content. Cement consisted of 40 percent portland clinker and 60 percent slag plus gypsum to provide the indicated SO_3 content. Fineness of cement—3793 sq cm per g (Blaine)

(Fig. 1). Belgian specifications allow up to 3 percent.[4] Lerch[7] has shown that it is necessary to add an adequate amount of gypsum to suit the composition and fineness of each portland cement, and that the optimum amount may exceed the 2.5 percent limit for Type I with a C_3A content higher than 8 percent. As in portland cement, the addition of the proper amount of gypsum to portland-blast furnace slag cement is of great importance.[3] A performance test, like the leach test devised by Lerch[7] should be established for both types of cement, with the following limits: SO_3 extracted at 18 hours —not less than 0.1 g per l, and at 24 hours not more than 0.5 g per l.

Physical tests

Table 3 shows the fineness, time of set and ASTM standard cube and briquette strength of the blended cement compared with straight portland cement made with clinker of the same composition. As can be seen, the substitution of slag for such large amounts of portland clinker does not impair the strength. Both cements 60 and 50 exceed by about 50 percent the ASTM C 205 strength requirements which are equal to those for commercial portland cement (ASTM C 150, Type I).

The specific surface is about 13 percent higher in the slag blends than in straight portland. This greater fineness is not difficult to obtain, and as the slag is less active than the portland clinker, finer grinding is required to facilitate its hydration. The strength of both blended cements at the age of one year is higher than that of portland cement.

Since the finishing tube mills that grind together the slag, the clinker and the gypsum are properly cooled by water sprayed on the outside shell, no gypsum dehydration occurs and the cement does not stiffen prematurely.[6]

Heat of hydration

Heats of hydration were determined by the heat of solution method (ASTM C 186-49) (Table 2). The low heat of hydration at 28 days of cement 60

TABLE 3—PHYSICAL TESTS OF CEMENTS

Test	Cements made with Monterrey clinker, Type II			ASTM requirements C 205-48T Portland-slag, Type IS
	Portland	Portland-slag		
		60	50	
Normal consistency, percent of water	24.0	26.0	25.3	——
Time of setting, hr: minutes				
Initial	3:17	3:55	5:07	Not less than 45 minutes
Final	6:05	6:25	8:17	Not more than 10 hr
Percent retained on No. 325 sieve	8.9	4.7	4.2	12 (max)
Specific surface, Blaine sq cm per g	3315	3750	3713	3400 (min)
Soundness, autoclave expansion or contraction, percent	0.03	−0.03	−0.06	0.20 (max)
Strength, psi				
Compressive				
3 days	1327	1612	1313	900 (min)
7 days	2475	2780	2572	1800 (min)
28 days	3883	4580	4452	3000 (min)
3 months	6706	5803	6680	
1 year	7396	7994	7834	
Tensile				
3 days	——	302	257	150 (min)
7 days	365	470	446	275 (min)
28 days	496	543	641	350 (min)

complies with the U. S. Federal Specification SS-C-192 for Type IV, low-heat portland cement, and exceeds the 7-day maximum limit by only 4 cal per g. The heat of hydration of cement 50 exceeds U. S. Federal requirements (SS-C-192) for Type II by only 7 and 5 cal per g at 7 and 28 days, respectively. Cement 60 is preferable for mass concrete construction.

By stipulating clinker composition of C_3S between 40 and 50 percent and C_3A lower than 8 percent (preferably lower than 6 percent), and by increasing the slag-clinker ratio, the heat of hydration can be controlled and reduced.

Satisfactory slag cement with low hydration heat was secured for the Vesdre Dam, Belgium (Fig. 2), by fine grinding and reducing the amount of clinker to only 15 percent.[4]

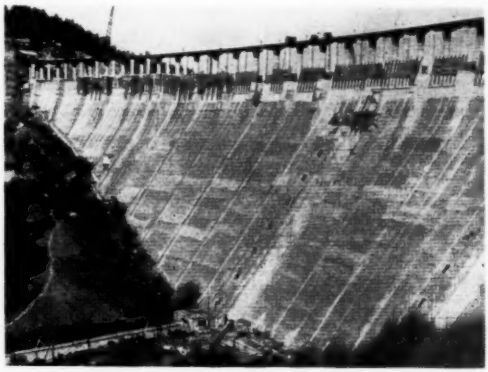


Fig. 2—Vesdre Dam, Belgium, built with portland-slag cement consisting of 15 percent portland clinker and 85 percent slag.

Sulfate resistance

Tests were made according to the U. S. Corps of Engineers method CCL-CEM-50. Mortar bars, 1:6 with standard Ottawa sand, cured for 14 days, were submerged in a solution containing 0.075 moles of Na_2SO_4 and 0.075 moles of $MgSO_4 \cdot 7H_2O$ per liter. For comparison, bars made with straight portland cements, ASTM Types I, II and V, were also included in the tests, together with portland-blast furnace slag cements 50 and 60. After 49 days immersion in the sulfate solution, the following expansions were measured.

Type of cement	*Expansion in percent of original length*
Straight portland, Type I	Broken by excessive expansion
Portland-slag 60	0.25
Portland-slag 50	0.15
Straight portland, Type II	0.09
Straight portland, Type V	0.05

The expansion of the slag cements in the sulfate solution was much less than that of Type I portland cement, and this fact may explain the European preference for slag cements for maritime work.

CORRECTIVE PROPERTIES OF SLAG AGAINST ALKALI-AGGREGATE EXPANSION

Mortar bars, 1 x 1 x 10 in., stored in sealed containers at 100 F according to ASTM Method C227-50T, were used in tests to determine the corrective properties of the slag against alkali-aggregate expansion.

The expression "alkali content" is here used to mean the sum of the percent of Na_2O in the total cement (clinker plus slag plus gypsum), plus 0.658 times the percent of K_2O in the same total cement, plus the Na_2O in the $NaOH$ added, computed as percent by weight of total cement used in the test.

Na_2O and K_2O were determined by flame photometer after the cement was dissolved in HCl (ASTM C228-49T). Gilliland and Bartley[9] found that, with time, all of the HCl soluble alkalies dissolve in water, that is, they become available to react with the aggregates. (At 7, 28 and 90 days, about 74, 87 and 95 percent, respectively, of the total HCl soluble alkalies, went into solution in water.)

Adding $NaOH$ to the mixing water to increase the alkali content in the tests does not give results equal to those when the high alkali content is already present in the cement because the Na_2O added as $NaOH$ is all in solution and available when mixing is started. The Na_2O contained in the cement becomes gradually available as the alkali-bearing cement compounds are hydrated. Nevertheless, such compounds hydrate quite promptly[9] and many investigators[10,11,12,13,14] have satisfactorily used $NaOH$ to raise the alkali content and thus cause a rapid attack on reactive aggregates.

Fig. 3 shows the results obtained by using crushed Pyrex glass as reactive aggregate after the method devised at the Bureau of Reclamation laboratories.[15,16,17] It was modified by adding $NaOH$ to the mixing water in varied proportions so that the same Na_2O equivalent (1.23 percent) was available in each case to react with the Pyrex glass and produce high expansion to be checked by the corrective material. Test *d* was left with low alkali for comparison.

For the same high alkali content the slag blends (*b* and *e*) show much less expansion than low-alkali portland with high $NaOH$ addition (*f*), or high-alkali portland with low $NaOH$ addition (*a*). Therefore it is logical to credit the reduction in expansion in the slag blends to the properties of the slag and not to a decrease in the alkali content due to the elimination of the cement replaced by the slag. According to Bureau of Reclamation Specification No. 3172 (Pozzolan for Cachuma Dam),[17] at the age of 14 days a minimum reduction in expansion of 60 percent is required from fly ash (considered a satisfactory pozzolan). Slag blends *b* and *e* compared with cement *f*, reduced expansion 68 percent at 13 days in container (14 days of age); higher than the minimum required for fly ash. This is considered satisfactory, especially when in the USBR specification the 25 percent fly ash replacement reduces the alkali content of the blend, whereas in this test the 50 percent slag replacement does not reduce the alkali content since $NaOH$ is added to keep the same percentage in the slag blends as in the straight portland taken as con-

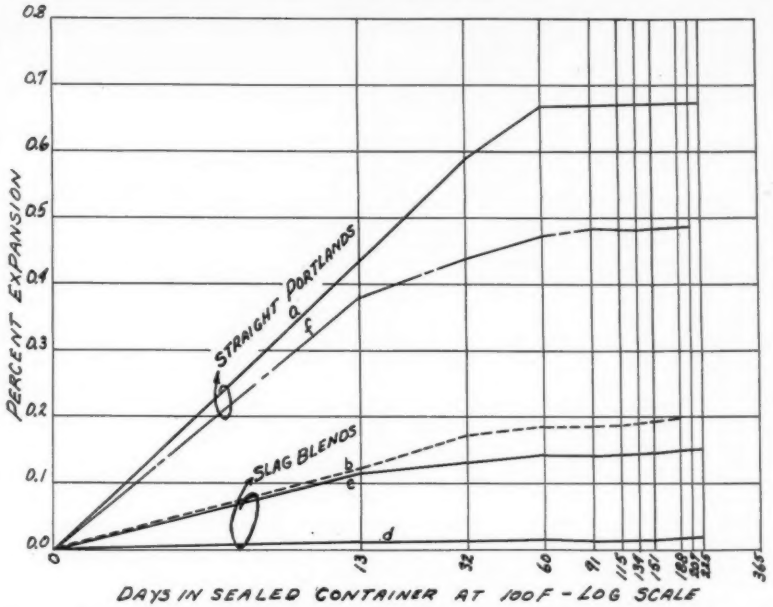


Fig. 3—Reduction in expansion by portland-slag cements. Aggregate—Crushed Pyrex glass sand, 1:2.25 mix. Cement *a*—Portland ASTM Type II, high alkali (0.77 percent). Cement *f*—Portland ASTM Type II, low alkali (0.25 percent). Cement *b*—Laboratory-blended 50/50 portland-slag plus gypsum to provide 3 percent SO_3 similar to ASTM C 205-IS, medium alkali (0.44 percent). Cement *e*—Portland-slag cement ASTM C 205-IS, low alkali (0.30 percent). To these cements $NaOH$ was added to raise alkali content to 1.23 percent. Cement *d*—Same as Cement *e* but with no $NaOH$ added.

trol. For the slag cement without $NaOH$ addition, test *d*, the expansion is negligible, as expected, since its alkali content is low (0.30 percent).

CONCLUSIONS

a. Due to its efficient hydraulicity, permitting a high percentage of cement replacement without sacrificing strength, and its low cost where it is obtained as a by-product from pig iron production, granulated blast furnace basic slag is considered an efficient means of preventing alkali-aggregate expansion. Its use should be carefully compared, in each case, with other possibilities, such as the adoption of a portland-pozzolan blend, which, in many instances, when good pozzolan is available, will be most satisfactory.[18] The lower cost of slag, that when available can replace 50 or 60 percent of the portland cement clinker, may favor its use in many other cases.

b. Due to its low-heat of hydration, portland-blast furnace slag cement, when economically available, should be considered as a possible substitute for portland cement Types II and IV. Slag blends cause drying contraction similar to that of many pozzolan cements, and about 50 percent higher than portland cements. Therefore, slag blends and pozzolan cements are more appropriate for thick hydraulic structures that never dry entirely, or for submerged structures, than for thin members subjected to drying conditions.

c. When so required, ASTM Specification C 205 can be supplemented with special re-

quirements to provide lower heat of hydration, or higher sulfate resistance, by fixing limits for the amount and composition of the clinker entering into the blended cement.

d. To insure the addition of the proper amount of gypsum to the portland-blast furnace slag cement, it is advisable that the SO_3 limit in ASTM Specification C 205 be replaced by a performance leach test, similar to the one devised by Lerch.

ACKNOWLEDGMENTS

Valuable suggestions have been received from Robert F. Blanks of the U. S. Bureau of Reclamation. The tests were made under the direct supervision of Manuel Donde Gorozpe, Head, Concrete and Cement Central Laboratory of the Hydraulic Resources Ministry of Mexico.

REFERENCES

1. Josephson, G. W., Sillers, F. and Runner, D. G., *Iron Blast-Furnace Slag—Production, Processing, Properties and Uses*, Bulletin 479, U. S. Bureau of Mines, 1949.

2. Lea, F. M. and Desch, C. H., *The Chemistry of Cement and Concrete*, Edward Arnold & Co., London, 1937, p. 268.

3. Forsén, L., "The Chemistry of Retarders and Accelerators," *Symposium on the Chemistry of Cement*, Stockholm, 1938, p. 393.

4. Van Wetter, L. and de Clercq, J., "Le Barrage-Reservoir de la Vesdre, a Eupen," *Annales des Travaux Publics de Belgique*, Feb. and Apr. 1948, p. 30.

5. ASTM *Specification* C 205 (see Definition of Granulated Blast-Furnace Slag).

6. Barona de la O, F., "Mexico's Requirements in Cement for Mass Concrete Projects," *Rock Products*, Apr. 1950, p. 136.

7. Lerch, William, "Influence of Gypsum on the Hydration and Properties of Portland Cement Pastes," *Proceedings*, ASTM, 1946, pp. 1252-1292.

8. Stanton, T. E., "Expansion of Concrete Through Reaction Between Cement and Aggregate," *Transactions* ASCE, V. 107, 1942, p. 54.

9. Gilliland, J. L., and Bartley, T. R., "Water-Solubility of Alkalies in Portland Cement," ACI JOURNAL, Oct. 1950, *Proc.* V. 47, pp. 153-160.

10. Stanton, T. E., "California Experience with the Expansion of Concrete Through Reaction Between Cement and Aggregate," ACI JOURNAL, Jan. 1942, *Proc.* V. 38, Fig. 3, p. 214.

11. Stanton, T. E., "California Experience with the Expansion of Concrete Through Reaction Between Cement and Aggregate," Author's Closure, ACI JOURNAL, Nov. Suppl. 1942, *Proc.* V. 58, Table C, p. 236-20.

12. Kelly, T. M., Schuman, L. and Hornibrook, F. B., "A Study of Alkali-Aggregate Reactivity by Means of Mortar Bar Expansions," ACI JOURNAL, Sept. 1948, *Proc.* V. 45, Fig. 7, p. 47.

13. McConnell, Duncan, Mielenz, Richard C., Holland, William Y. and Greene, Kenneth T., "Cement-Aggregate Reaction in Concrete," ACI JOURNAL, Oct. 1947, *Proc.* V. 44, Fig. 17, p. 115.

14. Mielenz, Richard C., Greene, Kenneth T. and Benton, Elton J., "Chemical Test for Reactivity of Aggregates with Cement Alkalies; Chemical Processes in Cement Aggregate Reaction," ACI JOURNAL, Nov. 1947, *Proc.* V. 44, Table 6, p. 205.

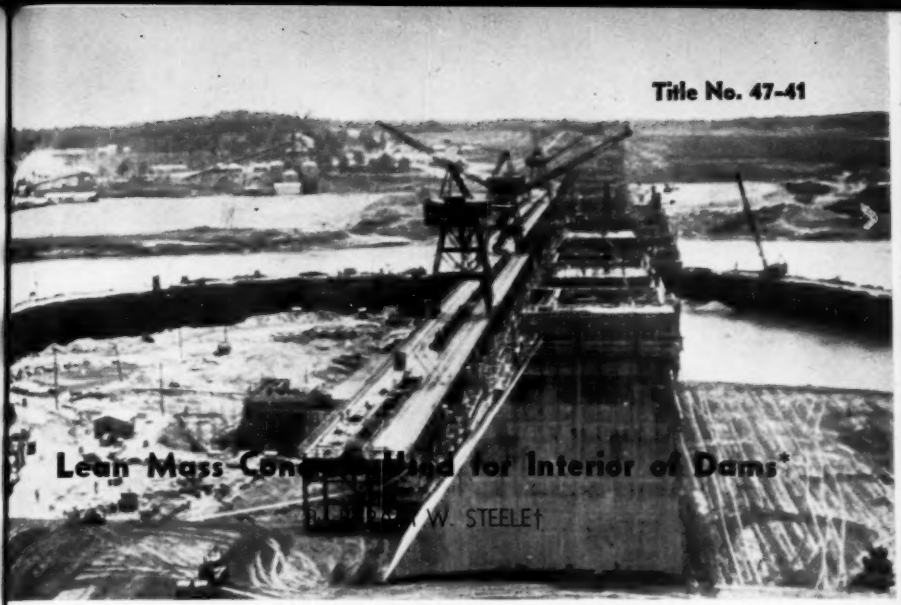
15. Gilliland, J. L. and Moran, W. T., "Siliceous Admixture Specified for Davis Dam Concrete," *Engineering News-Record*, Feb. 3, 1949, p. 64.

16. USBR *Specification* No. 1904, "Calcined Reactive Siliceous Material For Use in Concrete," Davis Dam, July 23, 1947.

17. USBR *Specification* No. 3172, "Portland Cement and Pozzolan or Portland-Pozzolan Cement for Cachuma Dam," Aug. 31, 1950.

18. *Symposium on Use of Pozzolanic Materials in Mortars and Concretes*, ASTM Special Publication No. 99, 1950.

Title No. 47-41



Lean Mass Concrete Used for Interior of Dams*

by W. W. STEELE†

SYNOPSIS

It is not necessary for concrete in the interior and exterior of a dam to have equal durability. By placing 4-bag concrete on exterior faces and lean mass concrete (2½-bag mix) in the interior, it is possible to achieve both durability and economy in gravity dam construction. Lean mass concrete is important in minimizing volume-change cracking; the ensuing economy is an additional feature. The use of interior and exterior mixes in Corps of Engineers dams is described.

INTRODUCTION

To be durable, concrete in a gravity dam must be relatively impermeable. If the concrete is not impermeable, water gradually seeps into and through it and dissolves and carries out chemicals, the loss of which contributes to deterioration.

However, to use sufficient cement to make the concrete throughout the entire dam as impermeable and as durable as the exterior shell must be to withstand severe weathering is uneconomical, and may be objectionable because of the influence of the hydration of the cement on volume change and attendant cracking.

It is uneconomical and unnecessary that concrete in the interior and exterior of a dam possess equal durability.

LEAN CONCRETE MINIMIZES VOLUME-CHANGE CRACKING

The initial temperature rise following the placement of each concrete lift in a dam depends upon the amount of cement per unit of volume, type of

*Presented at the ACI Washington Regional Meeting, Washington, D. C., Oct. 24, 1950. Title No. 47-41 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 22, No. 7, Mar. 1951, *Proceedings* V. 47. Separate prints are available at 35 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than July 1, 1951. Address 18263 W. McNichols Rd., Detroit 19, Mich.

†Member American Concrete Institute, Special Assistant, Civil Works Division, Office of the Chief of Engineers, Dept. of the Army, Washington, D. C.

cement, thickness of lift, and the time that elapses between placement of lifts.

If a sudden drop in temperature is experienced, or if the temperature gradient near the surface of the concrete is too steep, the tensile strength of the concrete may be exceeded and surface cracks develop, especially during severe storms following prolonged warm spells in the late fall and early winter. The size, number and location of these cracks are vitally affected by the ambient temperature conditions and by the temperature gradient between the surface and near-surface or interior of the mass. These cracks are the entering wedges for seeping water, frost action and the slow creeping deterioration that takes place through the many chemical and physical actions made possible by percolating water. Cracks once formed, however small, may later widen under the influence of falling temperatures and thus become a real menace. It is interesting to note that in Norway and Sweden the thin reinforced concrete in buttress-type dams has proved more durable than the concrete in gravity dams, principally because the surface cracking prevalent in gravity dams but not in buttress dams admits the acid waters so common in those countries, which in turn attack the concrete causing serious corrosion and making necessary expensive repairs.

From the standpoint of volume change and the possible attendant cracking, it would be desirable to use as low a cement content in massive concrete construction as is practicable to eliminate the early formation and later progressive opening of cracks. In lieu of a sufficiently low cement content, precooling of the concrete ingredients has become common practice in Corps of Engineers' dam construction as a second-best means of discouraging cracks. The precooling of ingredients for mass concrete is accomplished under four different systems—by the use of refrigerated water only in the mixing operations; by the use of refrigerated water and ice in the mixing operations; by the use of refrigerated water, ice and dunking the coarse aggregate in the refrigerated water; and by the use of refrigerated mixing water and air conditioned aggregate.

In discussing lean concretes emphasis should be placed on minimizing volume-change cracking rather than on economy. However, the possible economy should not be overlooked or disregarded as unworthy of consideration.

DEVELOPMENT OF LEAN CONCRETE

For many years engineers considered 4 bags of cement per cu yd of concrete as the minimum cement content permissible for construction of a dam. This was during the era in which strength was considered a measure of the ultimate durability. Not until strength began to be seriously questioned as an adequate measure of durability was there any concerted action taken to change the design criteria for mass concrete mixtures. In fact, it was claimed for a long time that it was not practicable to manufacture and place 6-in. cobble rock concrete having the proper characteristics with less than 4 bags of cement.

Gradually the attitude toward the 4-bag minimum for the interior of mass concrete dams began to change as mix design improved and the quality of concrete was brought to a higher degree of perfection by virtue of closer control, better gradation of aggregates and lower water-cement ratios.

About this time in the mass concrete mix redesign process, the value of air entrainment began to be appreciated as one of the most potent factors yet discovered in the improvement of concrete mix design. Within the past five years the manufacture of concrete has gradually been changed from a no-entrained air basis to the point where air entrainment is mandatory. Air entrainment has made possible more improvement in mass concrete than all other factors combined in the last 25 years.

"What is the minimum cement content permissible for use in the interior of a dam?" This is an extremely difficult question to answer until the materials economically available have been studied and tested in the kind of concrete that is to be used in the dam. It is not uncommon to find that several sources of aggregate are available for this type of construction and that all or many of these sources are of definite interest to contractors from a competitive bidding standpoint. Hence, all these available materials must be sampled, processed, concrete mixes designed and a careful laboratory study made to determine which materials offer the greatest advantage from the standpoint of results embodied in the finished concrete structure. Such a study will involve many months and, under certain conditions, a year or more may elapse before a fair appraisal can be made of the aggregates available. It is impossible to hurry investigations of this type and still obtain a satisfactory appraisal. Too little time allotted for such investigations has been the cause of many serious blunders in recent years and it will be the source of many more unless there is a determined effort to obtain the necessary information in ample time for use during the design stage of a project and well in advance of the construction stage.

One of the greatest hindrances to production of satisfactory lean mass concrete has been the nonuniform and poor gradation of fine aggregate and lack of proper equipment for producing the degree of uniformity and gradation required. Other industries have perfected means for producing the gradation required in their finished product, but this is not so in the production of fine aggregate for concrete.

For many years the writer has attempted to obtain a satisfactory appraisal of the value of gradation in fine aggregate relative to its effect on impermeability and placeability of mass concrete. Only within the past year has it been possible to obtain a satisfactory approach to such an evaluation and to prove definitely that considerable reduction in cement content is warranted if acceptable gradation in the sand can be uniformly maintained. It was proved in the construction of Grand Coulee Dam that sand of acceptable and uniform gradation could be produced at a reasonable cost from an entirely unacceptable raw material by breaking the raw material down into three fractions with hydraulic classifiers and recombining these fractions into a uniform, well-

graded product for a 4-bag concrete. But the range of 2- to 3-bag lean mass concrete now being successfully produced was not attempted. Sand is now being successfully produced with Dorrco hydraulic sizers by breaking the raw material down into several fractions (eight at one recent installation, five at another) and then recombining these fractions under a tight gradation specification and at a reasonable cost.

For many years the Corps of Engineers has been placing 3-bag concrete in the interior of dams where sand of acceptable gradation and uniformity could be obtained. This was accomplished without air entrainment, although it could have been much more satisfactorily accomplished with air entrainment, and the quality of impermeability would have been better. Conchas Dam interior 3-bag concrete with its average cylinder strength at one year of 3675 psi and its W/C of 0.69 by weight is an excellent example of what can be accomplished with 3-bag concrete in the interior of a dam. The interior concrete in Elephant Butte and Arrowrock dams, after 35 years, is excellent evidence of what can be accomplished with only 2 bags of cement per cu yd of concrete.

The relative cost of producing a sand of acceptable gradation and uniformity for the leaner concrete mixes has received unfavorable comment in the technical press in recent years, but regardless of this resistance from certain elements of industry, outstanding examples of initiative in the construction industry itself have proved conclusively that sand of acceptable gradation and uniformity for the production of impermeable lean mass concrete, having a cement content of 2 to 2½ bags per cu yd, can be produced at moderate operating costs and without excessive plant installation costs.

In current basic mixture design studies, conducted to explore the range of cement content permissible in lean mass concrete mixtures, it has been definitely proved that the finer sands, when combined with cement contents of 2, 2¼ and 2½ bags per cu yd, are entirely adequate from the standpoint of impermeability when compared with 3-bag mixtures that have been in use for many years. There is now satisfactory evidence that concrete at one year of age is decidedly less permeable than at three month's age. Still further reduction in permeability with time is expected in the specimens of this series, and there is reason to believe that the permeability of blends of portland cement and an active pozzolanic material will be still less permeable than the mixtures so far tested.

The permeability coefficient of concrete with a cement content of 2¼ bags per cu yd is not as low as the coefficient with a cement content of 3 bags per cu yd. Neither is the 3-bag permeability coefficient as low as for concrete made with 4 bags per cu yd. Such a comparison is not made here. This paper deals solely with lean mass concrete suitable for use in the interior concrete of a gravity dam.

The cement content of 2¼ bags per cu yd is not the minimum that could be used satisfactorily in the interior of a dam of moderate height and still produce a structure having adequate structural, impermeable and durable

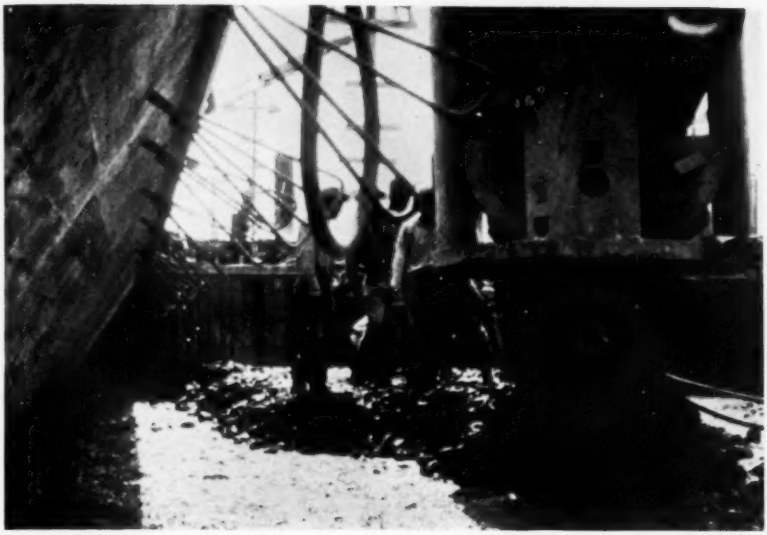


Fig. 1—Depositing the first row of interior mix concrete of a 5-ft lift near the downstream face of the dam. A row of interior 2½-bag batches is deposited entirely across the 50-ft monolith prior to placing exterior 4-bag concrete next to the downstream face. The ½-in. mortar layer which is spread on top of each lift immediately before placement of mass concrete has already been placed.

qualities, providing the water stored behind the dam is not corrosive to concrete as are the acid waters of Norway and Sweden. This entire development of lean mass concrete has been a step-by-step procedure in which the long used 4-bag mix was reduced to a 3½-bag mix; the 3½-bag to 3-bag; the 3-bag to 2¾-bag; the 2¾-bag to 2½-bag; and then the 2½-bag to 2¼-bag concrete; not on just one job with natural aggregate but on different jobs and with both natural and manufactured aggregates. The next step is 2-bag concrete, which was used successfully in Arrowrock and Elephant Butte dams.

It will take time to develop the absolute minimum cement content for lean mass concrete but the resulting economic saving would repay many times over the cost of the necessary basic research work. The quality of the resulting concrete rammed and tamped in place would be superior to the lean mass concrete product now being used and floated into its final resting place with powerful vibrators. The ramming and tamping would squeeze out surplus water and thus eliminate much of the defective underside bond in mass concrete.

INTERIOR VERSUS EXTERIOR MIXTURES

So that a better conception of the specific differences between the interior (2½-bag) and the exterior (4-bag) mixtures may be obtained, typical mix

Fig. 2—The first bucket of exterior 4-bag concrete being vibrated against the form. This view illustrates just how the exterior and interior mixes are placed and how they are vibrated into a solid mass without any danger of cold joints being experienced along the line of contact of the two types of mixes.

data for Buggs Island Dam is shown in Table 1 and illustrated in Fig. 1, 2 and 3. This is an aggregate manufactured from fine-grained granite and, hence, has all the disadvantages of a harsh sand and few of the advantages of a good water-worn, well-graded natural sand. The principal difference between the interior and exterior mixture is, of course, the cement content. The corresponding water content for the 2½- and 4-bag mixtures is 169 and 188 lb per cu yd. The actual job range in W/C is 0.69 to 0.73 for the 2½-bag mix and 0.47 to 0.50 for the 4-bag.

Entrained air is maintained in the upper range of the specification limit of 3 to 6 percent in the −1½-in. portion of the mixture. The higher values of entrained air do not affect the strength of lean mass concrete appreciably, but they do assist materially in concrete placement.

The ratio of fine to total aggregate varies with each job, some aggregates permitting as low as 0.21 or slightly less and others requiring as high as 0.26 or 0.27 to produce satisfactory results.

In the gradation of the sand, the fineness of the sand as batched into the mixer is 2.59 while the fineness modulus of the sand as found in the concrete after the mixing process is 2.21. The aggregate grinding action in the mixer

TABLE 1—MASS CONCRETE MIX DATA FOR BUGGS ISLAND DAM

Feature	Mix	
	Interior	Exterior
Coarse aggregate	Mfg. from fine-grained granite	
Fine aggregate	Mfg. from fine-grained granite*	
Bags of cement per cu yd of concrete	2.5	4.0
Slump, in.	0.5 to 1.0	1.0 to 2.0
W/C, gal per bag		
W/C by weight	0.69 to 0.73	0.47 to 0.50
Percent entrained air in minus 1½-in.	4.5 to 6.0	4.0 to 5.75
Ratio fine to total aggregate	0.23	0.21
Strength at 28 days, psi	1800 to 2650	4400 to 5300
Strength at 90 days, psi	2300 to 3500	4800 to 6300
Materials		
Portland cement, lb per cu yd	181	308
Natural cement, lb per cu yd	54	68
Water, lb per cu yd	169	188
Sand, lb per cu yd	837	729
No. 4 to ¾-in., lb per cu yd	509	486
¾ to 1½-in., lb per cu yd	546	555
1½ to 3-in., lb per cu yd	764	729
3 to 6-in., lb per cu yd	982	972
Sand gradation	As batched (typical)	After mixed (typical)
No. 4	0.0	0.0
No. 4 to No. 8	13.8	9.7
No. 8 to No. 16	19.2	14.3 | 24
No. 16 to No. 30	18.2	17.1
No. 30 to No. 50	20.3	21.5 | 59
No. 50 to No. 100	18.3	20.8
No. 100 to No. 200	6.1	10.5 | 17
No. 200 to pan	4.1	6.1
Fineness modulus	2.59	2.21

*Sand manufactured with rodmill.

Meticulous accuracy is required in batch plant performance. This is especially true in regard to weight of natural cement, since the air-entraining agent is interground with the natural cement.

Fig. 3—Another view of the vibration of exterior concrete after the interior concrete has been placed.

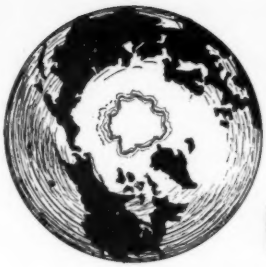
increased the proportion of the four −30 mesh sizes of sand and reduced the proportions of all other aggregate sizes, especially the three +30 mesh sizes of sand.

As the cement content per cubic yard is reduced, some form of fines is necessary to offset the effect on placeability of the loss of the cement particles. Hence, the mixer grinding action experienced with many aggregates is beneficial to lean mixtures by producing the necessary fines to offset the loss of

cement, providing it does not produce too much extremely fine material, as in the case of some limestones where excessive amounts of −200 mesh material are produced in the mixing process. Excessive extremely fine material affects the concrete mixtures adversely by increasing the water demand and by making the mixtures sticky and difficult to place rather than improving the placeability by producing a mixture that wilts readily under vibration and is easily molded. The latter result is accomplished with properly graded fines and a proper balance between the coarse, the intermediate and the fine fractions of the sand.

It has been observed on jobs where an excessive amount of −200 mesh material was being produced by mixer grinding action that 2½-bag concrete was more acceptable from the standpoint of satisfactory placement than 3-bag concrete and that still lower cement contents would be acceptable if the proper control of uniformity in the sand could be maintained.

Concrete is controlled from batch to batch and, hence, uniformity is the most important single factor in the control of concrete. Without a reasonable degree of uniformity, the optimum results sought in concrete, and especially in lean mass concrete, are not possible.



Letters from Readers*

BY WAY OF SYNOPSIS

R. A. Caughey and M. Hirschthal discuss the need for reports on the progress of prestressed concrete and the development of new techniques.

A design method for eccentrically loaded corner columns is explained by L. S. Muller.

K. P. Billner offers a graphic method for design of prestressed I-beams and box girders.

Data from tests of reinforced concrete beams is analyzed by Eivind Hognestad.

Experience with reinforced concrete pavements is reported by Charles P. Smith.

Andrew Brodsky discusses methods of mechanical prestressing.

Herbert A. Sawyer, Jr., adds to the discussion on elastic versus plastic theory.

Wanted: Guide in Developing Prestressed Concrete (LR 47-74)

Prestressed concrete, although used considerably in Europe, and now being considered seriously by American engineers, can still be classed in the development stage. The increased use of prestressing raises a question. What effects are patents going to have on the work of organizations who are interested in developing prestressed construction? When, for instance, means of holding rods are devised, will there be danger of infringing upon certain patents, present or future?

The above is only one question about this new construction method—there are many more. The writer would like to suggest that the American Concrete Institute take some definite steps to guide the development of this type of construction.

As matters now stand, there are many different notations and many different methods but a dearth of information on certain details of design. Engineers do not know what kind of steel to specify; what strength it should have; how much loss of stress they should expect. This information should come to the designer from the best possible source and that source is the American Concrete Institute.

In "Patents and Codes Relating to Prestressed Concrete,"† Curzon Dobell stated that it is not now advisable to write a code on prestressing. The writer agrees, but it does seem that ACI should publish progress reports on the best and latest practice. These reports would not be final but should outline the best practice to date and serve as an exchange and developer of ideas.

In the early days of reinforced concrete design, procedure was developed and guided by a few men who disseminated their ideas

*A part of copyrighted Journal of the American Concrete Institute, V. 22, No. 7, Mar. 1951, *Proceedings* V. 47. Separate prints of the entire Letters from Readers Section are available at 35 cents each. Address 18263 W. McNichols Rd., Detroit 19, Mich.

†ACI Journal, May 1950, *Proc.* V. 46, p. 713.

by publishing them in various media. The names of Taylor and Thompson, Talbot, Turneaure and Maurer are fresh in the minds of many engineers. They did not begin by writing codes but published "progress reports." The infant that we now hold in our hands should not be allowed to "grow up like Topsy" but ACI or some other responsible organization should direct that growth.

R. A. CAUGHEY, Professor of Structural Engineering, Dept. of Civil Engineering, Iowa State College, Ames, Ia.

The review of the science and art of prestressing reinforced concrete published in the June 1950 JOURNAL* is an excellent summary of the history of prestressing and practically exhaustive coverage of all the numerous aspects and methods developed in the comparatively short period in which the procedures have been actively used.

The writer is struck with the fact that there are no provisions in any of the codes for the allowable stresses in the cold drawn wire used in prestressing designs. In prestressed concrete construction there is an immediate necessity for such a provision as well as one for higher allowable stresses in high-yield reinforcing steel which probably will be developed further to meet the competitive situation.

The writer has attempted to interest some research centers in testing a prestressing method which has been in his mind for many years. This method does not entail the use of wire reinforcement, and is based on post-stressing the reinforced concrete after the concrete has set. It is particularly applicable to bridges.

The back wall is made of sufficient thickness and provided with the required reinforcement to withstand the pressure of jacks which will be applied to the ends of the slabs or beams with sufficient eccentricity to provide the required prestress (post-stress) with no tension in the top fibers of the concrete. As the concrete shrinks, the loss is taken up by turns of the screws of the jacks until this loss is compensated for. Allowance, of course, would have to be made for the deflection of the cantilever back wall.

While the writer has not completely thought through the detailed method of procedure, two alternative methods suggest themselves for the maintenance of this prestress during the life of the structure. One is to fill the space between the back wall and the end of the slabs or girders piecemeal with high strength concrete, with an addition of a nonshrinking admixture. The other is the expensive provision of maintaining the jacks in place, covering the space with properly designed and protected precast slabs which are removable for inspection and maintenance.

The advantages of this method of "prestressing" are the simplicity of application, the economy and the ability to use reinforcing bars (high yield) instead of numerous wires.

The writer hopes that some researcher will test this method, after which it could be developed in more detail.

M. HIRSCHTHAL, consulting engineer, New York, N. Y.

*Germundsson, Thor, "Prestressed Concrete Construction Procedures," ACI JOURNAL, June 1950, *Proc.* V. 46, p. 857.

Eccentrically Loaded Corner Columns (LR 47-75)

Simple nomographs for quadratic corner columns with the moment acting outside the planes of symmetry may give quick insight into the magnitude of stresses, if the resultant force remains within the kern (*i.e.*, for uncracked section).

The well-known expression for the concrete stresses

$$f_{c1,2} = \frac{P}{A}\left(1 \pm \frac{e_x}{r_x} \pm \frac{e_y}{r_y}\right)$$

where e_x and e_y are the eccentricities, r_x and r_y the kern distances in directions x and y, may be transformed with the notation of Fig. 1, with concrete cover 8 percent of concrete dimensions in both directions and

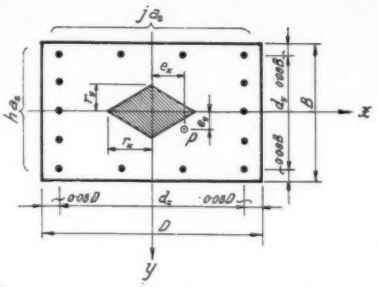


Fig. 1—Eccentrically loaded column

$n = 15$, assuming reinforcing bars of the same sectional area equally distributed along the periphery of the concrete section, by substituting for the bar distances from the axis of symmetry

$$\frac{d_x}{2} = 0.42D$$

$$\frac{j-3}{j-1} \times \frac{d_x}{2} = 0.42 \frac{j-3}{j-1} D$$

$$\frac{j-5}{j-1} \times \frac{d_x}{2} = 0.42 \frac{j-5}{j-1} D$$

and similarly in the y direction

$$\frac{d_y}{2} = 0.42\, B$$

$$\frac{h-3}{h-1} \times \frac{d_y}{2} = 0.42 \frac{h-3}{h-1} B \ldots \text{etc.}$$

If $A_c = BD$, $A_{sx} = ha_s = \mu_x A_c$ and $A_{sy} = ja_s = \mu_y A_c$ that is $\frac{\mu_x}{\mu_y} = \frac{h}{j}$,

$$A = \left[1 + 30 \frac{h+j-2}{h} \mu_x\right] A_c$$

Noting:

$$A = (1 + C_0\mu_x)\, A_c = R_0\, A_s$$

$$r_x = \frac{0.1667 + C_1\mu_x}{1 + C_0\mu_x} D = \frac{R_1}{R_0} D$$

$$r_y = \frac{0.1667 + C_2\mu_x}{1 + C_0\mu_x} B = \frac{R_2}{R_0} B$$

The values of C_0, C_1 and C_2 have been computed in Table 1 for two to 12 bars in both directions. The values of R_0, R_1 and R_2 may be taken from the two nomographs, Fig. 2 and 3. Assuming the variation of steel percentage between 0.8 and 8.0 percent, the limiting values of μ_x will vary according

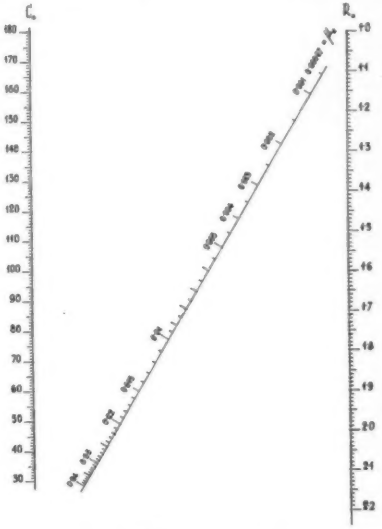


Fig. 2—Nomograph, C_0 and R_0

$$r_x = \frac{0.1667 + \frac{10.58}{h}\mu_x \left\{h + 2\left[\left(\frac{j-3}{j-1}\right)^2 + \left(\frac{j-5}{j-1}\right)^2 + \ldots\right]\right\}}{1 + 30\frac{h+j-2}{h}\mu_x} D$$

$$r_y = \frac{0.1667 + \frac{10.58}{h}\mu_x \left\{j + 2\left[\left(\frac{h-3}{h-1}\right)^2 + \left(\frac{h-5}{h-1}\right)^2 + \ldots\right]\right\}}{1 + 30\frac{h+j-2}{h}\mu_x} B$$

TABLE 1—VALUES OF C_0, C_1 AND C_2

↓ j	h →	2	3	4	5	6	7	8	9	10	11	12
2	μ_y/μ_x	1.000	0.667	0.500	0.400	0.333	0.285	0.250	0.222	0.200	0.182	0.167
	C_0 =	30.00	30.00	30.00	30.00	30.00	30.00	30.00	30.00	30.00	30.00	30.00
	C_1 =	10.58	10.58	10.58	10.58	10.58	10.58	10.58	10.58	10.58	10.58	10.58
	C_2 =	10.58	7.03	5.87	5.29	4.94	4.70	4.53	4.40	4.31	4.23	4.15
3	μ_y/μ_x	1.500	1.000	0.750	0.600	0.500	0.428	0.375	0.333	0.300	0.273	0.250
	C_0 =	45.00	40.00	37.50	36.00	35.00	34.26	33.75	33.33	33.00	32.72	32.50
	C_1 =	10.58	10.58	10.58	10.58	10.58	10.58	10.58	10.58	10.58	10.58	10.58
	C_2 =	15.87	10.58	8.51	7.40	6.70	6.21	5.84	5.58	5.35	5.19	5.03
4	μ_y/μ_x	2.000	1.333	1.000	0.800	0.667	0.571	0.500	0.444	0.400	0.364	0.333
	C_0 =	60.00	50.00	45.00	42.00	40.00	38.55	37.50	36.67	36.00	35.44	35.00
	C_1 =	11.70	11.35	11.15	11.03	10.98	10.93	10.83	10.82	10.80	10.80	10.80
	C_2 =	21.16	14.07	11.15	9.51	8.48	7.72	7.19	6.75	6.41	6.14	5.91
5	μ_y/μ_x	2.500	1.667	1.250	1.000	0.833	0.714	0.625	0.556	0.500	0.455	0.417
	C_0 =	75.00	60.00	52.50	48.00	45.00	42.84	41.25	40.00	39.00	38.16	37.50
	C_1 =	13.20	12.33	11.87	11.63	11.43	11.33	11.21	11.15	11.10	11.05	11.02
	C_2 =	26.45	17.60	13.80	11.63	10.21	9.23	8.50	7.92	7.47	7.11	6.80
6	μ_y/μ_x	3.000	2.000	1.500	1.200	1.000	0.857	0.750	0.667	0.600	0.545	0.500
	C_0 =	90.00	70.00	60.00	54.00	50.00	47.13	45.00	43.33	42.00	40.88	40.00
	C_1 =	14.80	13.40	12.68	12.25	11.98	11.78	11.61	11.50	11.41	11.35	11.25
	C_2 =	31.75	21.10	16.43	13.75	11.98	10.78	9.82	9.10	8.52	8.08	7.69
7	μ_y/μ_x	3.500	2.333	1.750	1.400	1.167	1.000	0.875	0.778	0.700	0.636	0.583
	C_0 =	105.00	80.00	67.50	60.00	55.00	51.42	48.75	46.67	45.00	43.60	42.50
	C_1 =	16.43	14.47	13.56	12.95	12.57	12.27	12.03	11.87	11.75	11.62	11.53
	C_2 =	37.00	24.65	19.07	15.85	13.75	12.27	11.15	10.27	9.58	9.02	8.57
8	μ_y/μ_x	4.000	2.667	2.000	1.600	1.333	1.142	1.000	0.889	0.800	0.728	0.667
	C_0 =	120.00	90.00	75.00	66.00	60.00	55.69	52.50	50.00	48.00	46.32	45.00
	C_1 =	18.10	15.60	14.39	13.62	13.10	12.75	12.45	12.27	12.10	11.97	11.83
	C_2 =	42.32	28.15	21.75	18.00	15.50	13.79	12.45	11.43	10.65	10.00	9.42
9	μ_y/μ_x	4.500	3.000	2.250	1.800	1.500	1.285	1.125	1.000	0.900	0.819	0.750
	C_0 =	135.00	100.00	82.50	72.00	65.00	60.00	56.25	53.33	51.00	49.04	47.50
	C_1 =	19.80	16.75	15.20	14.27	13.63	13.22	12.88	12.62	12.40	12.23	12.10
	C_2 =	47.55	31.70	24.40	20.05	17.25	15.25	13.80	12.62	11.72	10.95	10.30
10	μ_y/μ_x	5.000	3.333	2.500	2.000	1.667	1.428	1.250	1.111	1.000	0.910	0.833
	C_0 =	150.00	110.00	90.00	78.00	70.00	64.26	60.00	56.67	54.00	51.76	50.00
	C_1 =	21.55	17.83	16.01	14.95	14.20	13.70	13.30	13.02	12.78	12.58	12.40
	C_2 =	52.90	35.20	27.00	22.23	19.02	16.78	15.15	13.79	12.78	11.90	11.20
11	μ_y/μ_x	5.500	3.667	2.750	2.200	1.833	1.571	1.375	1.222	1.100	1.000	0.918
	C_0 =	165.00	120.00	97.50	84.00	75.00	68.54	63.75	60.00	57.00	54.48	52.50
	C_1 =	23.25	19.00	16.90	15.63	14.80	14.20	13.75	13.40	13.10	12.85	12.67
	C_2 =	58.15	38.80	29.65	24.35	20.80	18.30	16.42	14.97	13.81	12.85	12.05
12	μ_y/μ_x	6.000	4.000	3.000	2.400	2.000	1.714	1.500	1.333	1.200	1.090	1.000
	C_0 =	180.00	130.00	105.00	90.00	80.00	72.83	67.50	63.33	60.00	57.20	55.00
	C_1 =	24.95	20.20	17.80	16.35	15.40	14.70	14.18	13.77	13.43	13.17	12.97
	C_2 =	63.45	42.30	32.30	26.50	22.55	19.78	17.75	16.15	14.87	13.81	12.97

to the formula $\mu_x = \dfrac{h}{h + j - \frac{2}{2}} \times \dfrac{p}{2}$

from $\mu_{x\,min} = \dfrac{2}{2 + 12 - 2} \times 0.004 = 0.00067$

to $\mu_{x\,max} = \dfrac{12}{12 + 2 - 2} \times 0.04 = 0.04$

Example

The section shown in Fig. 4 is loaded by:

P = 280 kips M_x = 70 ft-kips
M_y = 42 ft-kips A_c = 427 sq in.
A_x = 6–¾-in. ϕ bars μ_x = 0.00623
A_y = 11–¾-in. ϕ bars μ_y = 1.833 μ_x = 0.0114
For h = 6 and j = 11 from Table 1:
C_0 = 75, C_1 = 14.8 and C_2 = 20.8

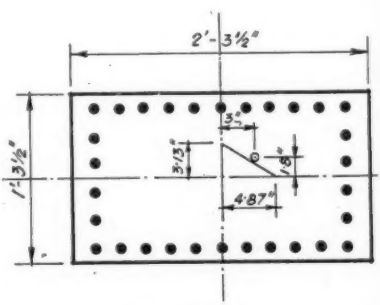


Fig. 4—Example

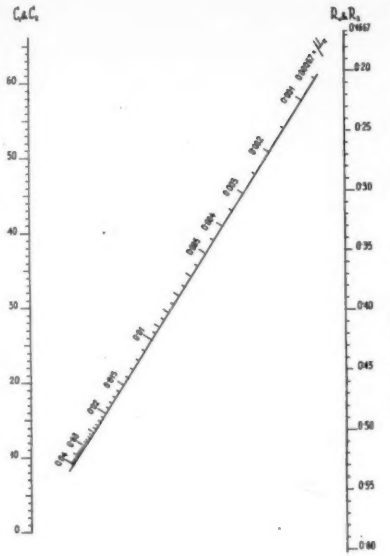


Fig. 3—Nomograph, C_1, C_2 and R_1, R_2

$$e_x = \frac{70 \times 12}{280} = 3 \text{ in.} \quad e_y = \frac{42 \times 12}{280} = 1.8 \text{ in.}$$

From the nomographs:

$R_0 = 1.47$, $R_1 = 0.26$ and $R_2 = 0.297$

$$f_{c1,2} = \frac{P}{A_c}\left(\frac{1}{R_0} \pm \frac{e_x}{R_1 D} \pm \frac{e_y}{R_2 B}\right) =$$

$$\left(\frac{1}{1.47} \pm \frac{3}{0.26 \times 27.5} \pm \frac{1.8}{0.297 \times 15.5}\right)\frac{280{,}000}{427}$$

$f_{c1} = 980$ psi (compression)

$f_{c2} = 86$ psi (tension)

$f_{s\,max} = 15 \times 86 = 1290$ psi (approximately)

L. S. Muller, Palestine Land Development Co., Jerusalem, Israel

Economical Cross Section of Prestressed I-Beams and Box Girders Determined Graphically (LR 47-76)

In the October 1950 Journal the writer described a simple way of figuring stresses in a prestressed rectangular beam.* This addenda shows how a simple graphic method can be applied for computing the stresses in more complicated shapes such as I-beams and box girders. Two assumptions are made:

(1) The stress distribution in the concrete from maximum to minimum diminishes along a straight line. This is a true assumption for prestressed concrete, which in flexure behaves as a homogeneous beam. Tests by the author and by others verify this fact.

(2) The ideal stress distribution is triangular. For a maximum dead and live load, the base of the triangle, which equals the assumed working stress, is in the top surface of the beam and its apex is in the bottom surface. This stress distribution is reversed for dead load only.

It has been suggested that for prestressed concrete a small amount of tension, not to exceed 10 percent of the compressive working stress, can be allowed and that this would lead to a saving in the required amount of reinforcing steel.

A quick analysis (deviating to an infinitesimal extent from the true condition to save space) will be made to show whether this is worthwhile.

Fig. 1 shows a diagram of such a stress distribution. The resisting moment which is the total compression in the concrete section times the distance from its resultant to the center of gravity of the reinforcing steel equals $\frac{9hf_cq}{10 \times 2} = M_r$

The resisting moment of concrete in tension is $\frac{hf_cq}{10 \times 2 \times 10} = M_{1r}$

M_{1r} divided by $M_r = \frac{hf_cq \times 10 \times 2}{10 \times 2 \times 10 \times 9hf_cq}$

$= 0.011$

*Billner, K. P., "New Prestressing Method Utilizes Vacuum Process," ACI Journal, Oct. 1950, *Proc.* V. 47, p. 161.

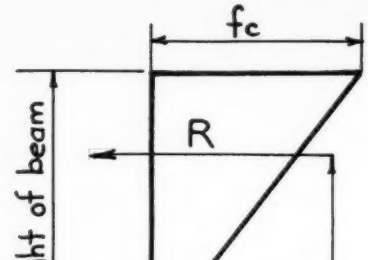


Fig. 1—Stress distribution in prestressed beam

The maximum steel saved by permitting tension in the concrete amounts to only 1.1 percent. Considering the possibility of cracks in the bottom surface, this steel saving is not worthwhile.

Graphic method for determining stresses in I-beam

f_c is the assumed working stress in the concrete.

Procedure—(1) Decide on the amount of working stress in the steel.

(2) Assume desired working stress in the concrete.

(3) By guess (and experience) decide on approximate cross section of concrete beam.

(4) Draw this cross section to a large scale and place the steel in such a manner as seemingly will result in best economy, *i.e.*, so that longest lever arm from center of gravity of steel to center of gravity of concrete compression zone is obtained.

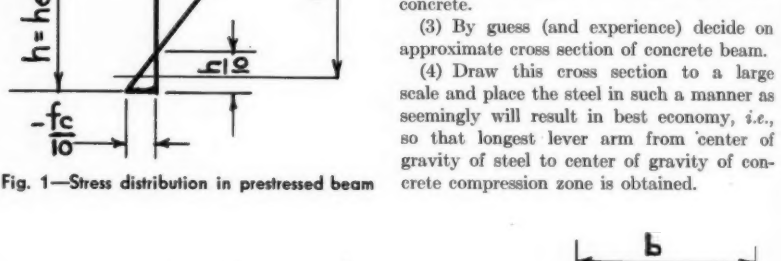


Fig. 2—Cross section of I-beam and stress distribution in concrete

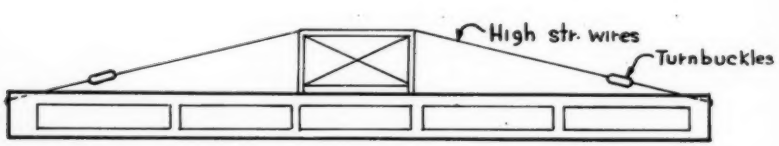


Fig. 3—To alleviate tension in top of beam prior to placement of deck, a light truss is attached to top of beam before prestressing

This type of beam generally carries a deck. The maximum moment, M, is then the sum of the dead load moment, M_1, from the weight of the beam and deck plus the live load moment, M_2.

Fig. 2b shows the stress distribution in the concrete. Calculate the total area of the figure between points 1-2-3-4-5-1 and multiply it by f_c. This is the total compression in the concrete when the beam is under full dead and live load.

Cut out this figure from a piece of cardboard. Hang it from a pair of calipers and adjust until the line 1-2 is vertical. This can be done by holding it near a wall. This establishes the center of gravity, *i.e.*, the location of the resultant, R. Scale the distance from R to the center of gravity of the steel. This is the lever arm, q.

If the experienced guess regarding the assumed cross section was correct (which is not likely) then $qR = M$.

If not, then retain the height of the beam but modify the width or thickness of the flanges. The desired thickness of the web should be established at the outset. Then try again; two or three tries should be sufficient to establish the concrete section.

The total steel area required, A_s, is found from the equation $A_s q f_s = M$, in which q, f_s and M are known.

Concrete stresses from dead load moment

$R_1 = R$ for a beam of symmetric cross section. For the ideal condition the dead load moment, M, would equal $A_s f_s q_1$. q_1 can be scaled; f_s and A_s are known.

Because the deck is not in place when the beam is being prestressed, it will be found that for this type of beam, which generally carries a live load amounting to several times the dead load, that this last equation is incorrect and that under dead load of the beam alone, tension develops in the top of the concrete. This could be overcome by placing steel higher up in the concrete, but this would be highly uneconomical because it would lead to a much higher percentage of steel and also require a larger concrete cross section.

This situation can be remedied without adding steel or concrete as shown in Fig. 3. Attach a light truss to the top of each beam before prestressing. It will be found that only a few wires are needed for the top cord. The number of these wires can be ascertained by establishing the center of gravity of all the reinforcing steel including these wires at the same elevation as that of R_1 in Fig. 2b.

Prestress these wires simultaneously with the wires in the beam and to the same extent. Only a few of these temporary trusses are required. They are available for re-use as soon as a substantial portion of the deck is in place. This is particularly simple if the deck is precast.

The design of a box girder is identical to that of an I-beam except that there are two webs instead of one.

With fair draftsmanship this graphic solution will give the correct answers within a tolerance of 1 percent. This is almost too exact when considering the irrational behavior of one of the elements with which we are dealing, *i.e.*, concrete.

K. P. Billner, President,
Vacuum Concrete, Inc.,
Philadelphia, Pa.

Concrete Beams with Compression Reinforcement (LR 47-77)

The Division of Building Statics and Structural Engineering at the Royal Institute of Technology, Stockholm, Sweden, has done some outstanding experimental work pertaining to reinforced concrete. It is felt that their *Bulletin* No. 4, "Concrete Beams with Compression Reinforcement," by A. Johnson, which also appeared in *Betong*, No. 1, V. 35, 1950, pp. 113-125, should be brought to the attention of ACI members.

Sixteen reinforced concrete beams were tested to study the effectiveness of compression reinforcement. All beams were designed to fail in compression and were provided with 4.6 percent tension reinforcement consisting of deformed bars with a yield point of about 96,000 psi. Concrete having a compressive strength of about 3000 psi was used in all beams. The specimens were about 4 x 6 in. and were tested as simple

TABLE 1—STUDY OF TEST DATA

Beam No.	b, cm	d, cm	d', cm	f'_{yp}, t per sq cm	A'_s, sq cm	f'_{cu}, kg per sq cm	Ultimate moment, M			Notes
							Test t × m	Calc. t × m	Test / Calc.	
1	10.0	12.8	——	——	0	221	1.46	1.27	1.15	
5	10.2	13.2	——	——	0	228	1.32	1.40	0.94	
9	10.1	13.3	——	——	0	214	1.47	1.33	1.10	
13	10.4	13.5	——	——	0	226	1.50	1.49	1.01	Stirrups
2	10.2	13.7	12.0	3.78	0.57	241	1.94	1.84	1.05	
6	10.0	13.4	11.7	3.62	0.57	218	1.57	1.59	0.99	
10	10.2	13.4	11.0	3.62	0.57	247	1.76	1.78	0.99	
14	10.1	13.4	11.7	3.62	0.57	209	1.57	1.55	1.01	Stirrups
3	10.5	13.0	11.2	3.24	1.57	249	2.01	2.06	0.98	
7	10.1	13.4	11.7	3.24	1.57	222	2.07	1.96	1.06	
11	10.0	13.3	11.6	2.74	1.57	227	1.83	1.85	0.99	
15	10.0	13.3	11.2	2.74	1.57	229	1.85	1.84	1.01	Stirrups
4	10.0	13.3	11.9	2.96	3.40	219	2.30	2.46	0.93	Failed in diagonal tension
8	9.9	13.0	11.6	3.01	3.40	215	2.02	——	——	
12	10.0	13.3	12.0	3.02	3.40	215	2.47	2.46	1.01	
16	10.2	13.5	11.3	2.99	3.40	214	2.16	2.45	0.88	Stirrups
								Average =	1.007	

beams over a span of about 11 ft 6 in. with two concentrated loads spaced about 20 in. The amount of compression reinforcement varied from 0 to 2.5 percent and consisted of plain bars of intermediate steel, 6 to 12 mm (¼ to ½ in.) in diameter. A unique feature was that no stirrups were provided in the majority of beams to secure the compression reinforcement against buckling. Only one beam of each type had vertical 3-mm (⅛-in.) stirrups spaced about 4 in.

The characteristics of the test specimens and the observed ultimate moments are presented in Table 1. Johnson used an approach similar to that advocated by R. Saliger for inelastic flexural analysis. This approach leads to simpler mathematical expressions than the standard straight line theory, and good agreement with the experimental data was obtained. The writer feels, however, that C. S. Whitney's analysis has definite merit in its striking simplicity, and he has therefore studied Johnson's experimental data in the light of Whitney's approach. The results of this study are presented in Table 1.

For rectangular beams failing in compression, Whitney has suggested the following expression for the ultimate moment:

$$\frac{M}{bd^2} = kf'_c + \frac{d'}{d}\, p' f'_{yp} \quad \ldots\ldots\ldots\ldots (1)$$

in which:

M = ultimate moment
b = width of rectangular section
d = effective depth of section
k = a constant equal to ⅓ for 3000 psi $< f'_c <$ 6000 psi. For values of $f'_c <$ 3000 psi, k is larger
f'_c = compressive strength of 6 x 12-in. concrete cylinders
d' = distance between centroids of compression and tension reinforcement
p' = percentage of compression reinforcement
f'_{yp} = yield point of compression reinforcement

Since concrete strength in the present tests is reported in terms of 6-in. cubes, f'_{cu}, and to correct for the concrete displaced by the compression reinforcement, Eq. 1 was modified as follows:

$$\frac{M}{bd^2} = k_1 f'_{cu} + \frac{d'}{d}\, p' (f'_{yp} - f'_{cu}) \ldots . (2)$$

The constant k_1 was computed by means of Eq. 2 from the test data. Values from 0.403 to 0.274 with an average of 0.347 were found. The calculated values of M in Table 1 were then obtained using the average value $k_1 = 0.347$.

The data in Table 1 refer to only one concrete with an average cube strength of 3200 psi. Certain conclusions may nevertheless be based on the table. The average ratio of observed to computed ultimate moment, 1.007, may not be interpreted as evidence in favor of Eq. 2, since the constant k_1 was computed from the test data. However, since the individual ratios for the

various beams vary relatively little from unity, this strongly indicates that the effect of compression reinforcement in reinforced concrete beams failing in compression is properly reflected by Eq. 2. The compression reinforcement adds its full yield point times the internal moment arm, d', to the moment capacity of the beam.

Since only beams 13, 14, 15 and 16 were provided with stirrups to secure the compression reinforcement against buckling, it may be concluded that such stirrups do not seem to be necessary to develop the full yield point of such compression reinforcement. Buckling of the reinforcement took place after the maximum moment had been reached regardless of the absence or presence of stirrups. The load carrying capacity of the beams without stirrups did, however, drop more rapidly after failure than in the beams provided with stirrups. Consequently, the impact absorbing capacity of such beams is probably improved by proper stirrups.

It may finally be noted that if Whitney's value of $k = \frac{1}{3}$ is used, and $f'_c = 0.9\, f'_{cu}$ is assumed, a value $k_1 = 0.30$ results. Hence, the ultimate moment capacity of the present beams will be predicted by Eq. 1, on the average, about 10 percent on the safe side. This equation further considers the effect of the quality of compression reinforcement and is in this respect, as well as in its simplicity, superior to the standard straight line theory of design.

EIVIND HOGNESTAD, Research Assistant Professor of Theoretical and Applied Mechanics, University of Illinois, Urbana, Ill.

Pavement Reinforcing Pays Off (LR 47-78)

The question of reinforcing concrete pavements continues to hold the attention of engineers. Experience gained from the municipal improvement program at Orange, Texas, has demonstrated to the writer the value of reinforcement in concrete pavements, both in city streets and state highways.

The city is 4 to 12 ft above mean low gulf level, being on tidewater on the Sabine River and inland from the Gulf of Mexico. With an annual rainfall of 50 to 80 in., the water table is continuously high. The ground has some areas of heavy black soil, some treacherous "gas mounds" of finely divided material, but mostly a thin topsoil over 6 to 12-ft of sandy clay with water-bearing sand under the clay. Most of the business section was originally a bayou and tidal marsh which was filled, 30 to 40 years ago, with 2 to 6 ft of sand pumped from the river and deposited on the swamp muck—certainly not ideal subgrade conditions.

The project started in 1939 and finished in 1940 included 15 miles of paving, 40, 36 and 31 ft back to back of integral roll curbs, and about 1 mile of 20-ft slab.

About three-fourths of the streets were 6 in., the balance 5 in., no thickened edges (except the effect of the integral curbs) and no thickened joints. The mix was carefully controlled, using 4½ bags of cement per cu yd. About 2 lb of local "blow sand" per cu ft of concrete was added to stop bleeding.

Reinforcement was a ⅜-in. round bar every 30 in. in two directions, with ⅜ in. x 4 ft dowels, 4 ft center-to-center on longitudinal center joints. The latter were spaced every 100 ft with two intermediate dummy joints; standard reinforcement was continuous under the dummy joints.

In 1940 Orange got its first $100,000,000 ship building contract and the jump in population required additional paving in new subdivisions. Federal housing agencies built 8 miles of concrete paving, partly on new hydraulic fill on the marginal swamps, partly on old ground; in addition about 4 miles of concrete access roads were built by the state.

There was no steel in 90 percent of the housing agency pavement and most of that 90 percent is now a total loss, past any possibility of repair or resurfacing.

The state-built access streets had edge steel only. There is a lot of cracking in these slabs and a great many of the joints are spalled; some failures have already been replaced.

The 10 to 11 year old town-built pavement with about 3 lb of steel per sq yd is in much better condition than the 5 to 7 year old pavements built by the housing and state agencies. Except for utility cuts and a couple sinks over sewer failures, there have been no repairs on the older pavement. In fact, some of the 25 year old concrete in the business district, which was reinforced with light mesh, is in as good condition as some of the nonreinforced 7 year old access streets.

CHARLES P. SMITH, civil engineer, Orange, Texas

Mechanical Prestressing (LR 47-79)

K. P. Billner is to be commended for his paper on the manufacture of a prestressed beam utilizing the vacuum process.* The method employed is relatively simple and economical, especially where many beams of identical cross section are required.

The writer would like to give recognition to Henri Lossier, who as early as 1944 used a method of mechanical prestressing.† Lossier made girders of two separate parts, connected by reinforcement. This reinforcement was prestressed by jacking up the parts. A stabilizing wedge was then inserted and the bars grouted. This method has the advantage of requiring no special anchorage or tensioning device.

ANDREW BRODSKY, professional engineer, New York, N. Y.

Skepticism Regarding the Plastic Theory of Design (LR 47-66)‡

H. M. Hadley's letter in the November JOURNAL is of value in emphasizing the limitations of "plastic theories" of flexure for concrete not reinforced in compression—limitations which were not emphasized in the original presentation of these theories. However, the writer cannot agree that these limitations are enough to place these theories in a class with the theory of the earth's flatness. These theories essentially retain their original value and the writer believes that this value is great.

Traditionally, the profession has always attempted to base its theories of flexural strength primarily on the behavior of axially loaded test specimens and the theory of elastic flexure. Discrepancies between actual and theoretical beam strengths have been recognized almost as long as reinforced concrete has been used.§ However, engineers have not been too concerned with these discrepancies because: (1) they have generally been on the safe side; (2) axial tests are easier to perform, duplicate and standardize than flexural tests; (3) rational theory inspires confidence and allows extrapolation, however unjustified these may be; and (4) traditional theory gives the strength as governed by steel, which is usually critical in practical design, more accurately than strength as governed by concrete.

Actually, elastic theory has been modified to agree more closely with beam test results by using an allowable elastic stress as high as 0.45 of the short-term, no-fatigue ultimate and a value of n higher than indicated by short-term cylinder tests.

The distinctive contribution of Whitney, Saliger and Jensen is their development of empirical relationships primarily based on beam test data, their (at least inferred) contention being that experimental beams mean more than cylinders in determining actual beam strength. Of course their formulas, like elastic theory formulas, are based on the principle that force times distance equals resisting moment. But a coefficient is always introduced which has not, as Hadley shows,

*"New Prestressing Method Utilizes Vacuum Process," ACI JOURNAL, Oct. 1950, *Proc.* V. 47, p. 161.
†Lossier, Henri, "Cements with Controlled Expansions and Their Applications to Prestressed Concrete," *The Structural Engineer*, Oct. 1946, p. 505.
‡See also ACI JOURNAL, Nov. 1950, *Proc.* V. 47, p. 257; Jan. 1951, *Proc.* V. 47, p. 398 and Feb. 1951, *Proc.* V. 47, p. 485.
§Talbot, A. N., "Tests of Reinforced Concrete Beams," ASTM *Proceedings*, V. 4, 1904, p. 542.

been justified by cylinder tests. These coefficients are determined solely to make the formulas conform to beam tests.

These coefficients can never be justified by cylinder tests unless some of the neglected variables which cause beams to behave differently from cylinders are evaluated. These variables include time, size, stress gradient, multi-axial stresses and restraints, placement and curing, and amount and location of reinforcing steel. Time is probably the most important of these variables.* Hadley emphasized this importance by his report of the cylinders whose stress-strain curves were radically altered by sudden increases in the rate of applying strain. Even if time is the only variable considered, no cylinder stress-strain curve, for which time must be variable, could ever represent stress distribution in a beam at any instant.

Engineers will never care much about all the variables necessary to reconcile cylinder and beam stress curves because the resulting relationships would be hopelessly complicated and completely useless for design.

The only requirement for validity in the empirical approach is that the ultimate resisting moment equal that given by beam tests, and for practically this moment should be expressed in terms of axial strengths of materials. Two requirements may be substituted for this basic requirement: first, that the ultimate tensile and compressive forces be those found from beam tests; and second, that the moment arm between these forces be that found from beam tests. *Any stress distribution pattern may be assumed which will satisfy these two requirements.* Obviously, any shape can be made to satisfy these requirements, such as triangular, rectangular, trapezoidal, parabolic, or hyperbolic. The shapes actually assumed were purely arbitrary. Any one of these authors could have selected the traditional triangular shape, and, by making its dimensions the proper functions of their coefficients, could have developed precisely the same beam strength formulas that they developed for their assumed shape. Except for simplicity, it made no practical difference which shape these authors selected. Agreement with cylinder stress-strain curves was unimportant and even undesirable because these curves are not correct for beams.

Therefore, the claim that these theories are wrong because their arbitrary stress distributions do not agree with the cylinder stress-strain curve is meaningless, except that it does expose the equally meaningless attempts of these authors to justify their results by claiming such agreement. These attempts made what was actually an arbitrary selection look like a basic and questionable assumption. Whitney recognized this for his method, stating that he did "not mean that this method is based on the assumption of rectangular stress distribution . . . It would be possible to use any curved shape that would give the same area and center of gravity."†

A simple expression, giving force and moment arm in terms of cylinder strength and coefficients which are ruthlessly and finally adjusted to agree with beam tests, is all that is ever likely to be useful for concrete design.

Whitney succeeded remarkably well in developing such a set of simple design formulas. He eliminated as many of the variables as possible and yet conformed to test results over a fairly wide range. His relationships obviously cannot be used for extreme strengths or other extremes in variables, because his empirical coefficients were not evaluated from beam tests for these extremes.

The present modified elastic theory satisfies these requirements as well as the handicap of the assumption of elasticity will permit, which is fairly well if it can be agreed that concrete should have a higher factor of safety than steel. But plastic theory has proved indispensable for columns and beams with compressive reinforcement, and the awkwardness of using elastic theory for bending in these members is a serious disadvantage which the Whitney theory avoids.

However, the accumulated experience of the profession, associated with the elastic theory as it is, would fade with a change in theory. The profession will decide in the long run whether a change is justified. The writer agrees with Hadley that the modified

*Shank, J. R., "Plastic Flow of Concrete at High Overload," ACI JOURNAL, Feb. 1949, *Proc.* V. 45, p. 493.
†*Transactions*, ASCE, No. 107, 1942, p. 259.

elastic theory is useful and that there need be no great urgency in discarding it.

The Jensen relationships were evidently not intended for design, but for accurate expression of ultimate strengths. Thus, these relationships conform better to test results (for large and uneconomical percentages of steel) because they use the additional, complicating variable, β. Values of β are determined, not from cylinder test results, but to make the Jensen relationships conform to beam test results.* Thus, these relationships do little more than the Whitney relationships to reconcile cylinder and beam data, and what little is gained in accuracy is lost in simplicity.

Saliger states† that the stress-strain-time curve for the standard prismatic specimen (size not specified, but probably 8 x 8 x 16 in.) has the same form as the stress-depth curve for the compression region of a beam, and also that the maximum stress for both curves has the same value, σ_p. He admits that the ultimate strains of the axial specimens average about 0.003, while those of beams average about 0.005. He also states that the standard cube (probably 8 x 8 x 8 in.) strength, σ_w, is not the "true" strength, σ_p, but that he has found σ_p to be 0.70 to 0.85 of σ_w.

But why should the strength of the 8 x 8 x 16 in. specimen be the "true" strength, and not the strength of a $2\frac{1}{2}$ x $2\frac{1}{2}$ x 5 in., 12 x 12 x 24 in., or 8 x 8 x 8 in. specimen, all of which Gonnerman‡ and others have shown to be appreciably different? Saliger does not tell us, but the answer must be that Saliger's "true" strength, σ_p, is true only because when it is used in Saliger's formulas it gives the right answers—the answers found by beam tests. Since Saliger offers no beam (or other) test evidence to show how he determined $\sigma_p = 0.70$ to $0.85\ \sigma_w$, his paper by itself is incomplete, and its basic validity is unproved—in contrast to the papers of Whitney and Jensen. Of course, Saliger's paper can only be fully judged in the context of his other writings, which the writer is not in position to do. Probably Saliger determined this relationship the same way he did his maximum strain—from beam tests and not from cylinder-type tests, despite his omission of this most vital evidence.

As for Hadley's criticism, the practical validity of Saliger's beam strength formulas does not depend on the shape of his assumed stress distribution. It depends on the success of his empirical determination of a coefficient to give the right answer when used with the assumed shape. The shape is only of academic interest; the vertical scale may be adjusted to make *any* shape give the correct moment arm, and the maximum stress may be adjusted to make *any* shape give the correct compressive force.

Hadley deserves credit for showing some of the weak points in these attempts to make cylinder test data fit beam test data. Our conclusion must be that we do not yet know enough about concrete to rigorously reconcile the two bodies of data (nor is this reconciliation of great practical value). But we certainly should not therefore neglect beam test data in our beam design. Nor should we, therefore, discredit the semi-empirical formulas which are basically derived from this data, whether they are called "plastic," "modified elastic," or otherwise.

HERBERT A. SAWYER, JR.,
Associate Professor of Civil Engineering, University of Connecticut, Storrs, Conn.

*Jensen, V. P., "The Plasticity Ratio of Concrete and Its Effect on the Ultimate Strength of Beams," ACI JOURNAL, June 1943, *Proc.* V. 39, p. 580, Fig. 14.
†*Beton und Eisen*, V. 35, 1936, p. 318.
‡*Proceedings*, ASTM, 1925, Part II, p. 237.

Current Reviews*

of Significant Contributions in Foreign and Domestic Publications

Development work at the Field Test Unit

Magazine of Concrete Research (London), No. 4, 1950 pp. 35-37

Construction and testing of pre-tensioned prestressed concrete joists under consideration for use in small home construction are described. Results of dead load tests 4½ times design load are reported. Details of the molds for casting the joists and the grips developed to prestress the wires are given.

Facts about some old Danish concrete roads (Oplysninger om nogle gamle danske betonveje)

Beton-Teknik (Copenhagen), V. 16, No. 4, 1950, pp. 131-136

AUTHOR'S SUMMARY

Some of the oldest Danish concrete roads are illustrated. For each road, one of the responsible officials gives a short statement regarding it from a technical as well as from an economic point of view.

The measurement of vibrations in freshly placed concrete

A. C. WHIFFIN, S. A. H. MORRIS and R. T. SMITH, *Magazine of Concrete Research* (London), No. 4, 1950, pp. 39-46

AUTHOR'S SUMMARY

Electronic apparatus and techniques for measuring vibrations are described. The equipment was used to measure the movements produced in freshly placed concrete by an experimental vibrator operating on the surface. Details are given of the circuits, calibration arrangements and the type of information obtainable.

The preparation of concrete aggregate

R. E. POCHIN, *The Reinforced Concrete Review* (London), V. II, No. 3, July 1950, pp. 149-170

Reviewed by C. P. SIESS

The author discusses in a general almost nontechnical manner the processes and problems relating to the production in England of concrete aggregates from both crushed stone and gravel. Topics discussed include quarrying, crushing, washing, screening, plant layout, and material handling.

Strength variations of hydrated binders (Sur les variations de resistance des liants hydrates)

L. CHASSEVENT and D. DOMINE, *Revue des Materiaux* (Paris), Special Issue, Oct. 1950, pp. 44-7

Reviewed by PHILLIP L. MELVILLE

Strength variations of hydrated plaster and portland cement were studied as a function of time. Flexural strengths were recorded as samples were kept immersed. Strengths were reduced and apparent volumes increased with treatment.

Multistory rigid frames

K. HRUBAN, Brno-Prague (Czechoslovakia) 1950, 304 pp.

Reviewed by IVAN M. VIEST

This book contains the theory, description of details, and tables of moments for the design of multistory rigid steel and reinforced concrete frames. The main body of the book is based on elastic theory. One chapter is devoted to a discussion of the redistribution of moments in reinforced concrete structures which takes place at high loads. Tests of four single-bent frames made by the author at the Technical University in Brno are described.

*A part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 22, No. 7, Mar. 1951, *Proceedings* V. 47. Address 18263 W. McNichols Rd., Detroit 19, Mich. Copies of articles or books reviewed are **not** available through ACI. In most cases they can be obtained direct from the original publishers. Address, when available, will be furnished by ACI on request.

Colour and texture in concrete surfaces

J. G. WILSON, *The Reinforced Concrete Review* (London), V. II, No. 3, July 1950, pp. 171-188
Reviewed by C. P. SIESS

The article is confined to surface treatments for precast concrete facing units. Types of treatments considered include exposed aggregate, tooled surfaces, acid treatment, ground surfaces, and the use of textured mold linings. There are several illustrations.

Considerations in testing floor construction (Worauf muss bei der Durchfuehrung von Deckenversuchen geachtet werden?)

K. GAEDE, *Betonstein-Zeitung* (Wiesbaden), V. 1950, No. 7, pp. 155-159
Reviewed by RUDOLPH FISCHL

This paper demonstrates how to make large scale tests of floor construction outside the laboratory. Size and arrangement of test load, choice of loading points in relation to maximum moment and shear, proper dimensions of test members, span and support, number of tests, properties of the building material and observation of deformations during the test are discussed.

Explanatory handbook on the British Standard Code of Practice for Reinforced Concrete

W. L. SCOTT, W. H. GLANVILLE and F. G. THOMAS, Concrete Publications Ltd., London. 118 pp. $2.10

This book interprets the British Standard Code of Practice for the Structural Use of Normal Reinforced Concrete in Buildings adopted in 1948. This new code is now commonly used in Britain in the design of reinforced concrete structures. All the clauses requiring explanation are discussed so that the intention is made clear and many tables are included to aid design according to the code which is given in its entirety interspersed with the necessary comments.

Ultimate design of concrete structures

K. HRUBAN, Brno-Prague (Czechoslovakia), 1949, 184 pp.
Reviewed by IVAN M. VIEST

The ultimate design of structural elements made of reinforced concrete was introduced into the Czechoslovakian design practice by the specifications CSN 1090-1948—Supplement. In this book, Professor Hruban discusses the theoretical and experimental basis for these specifications and in several numerical examples gives the designer a guide for practical application of the newly introduced principles. The book contains a detailed English summary.

New method of measuring hydration of hydraulic binders (Nouvelle methode de mesure de l'hydration des liants hydrauliques)

M. REY, *Revue des Materiaux* (Paris), Special Issue, Oct. 1950, pp. 39-43
Reviewed by PHILLIP L. MELVILLE

Since Le Chatelier the reaction of water on hydraulic binders has been studied by physical and chemical means. This study covers the effect of temperature from 2 to 80 C using a device to record isothermic volume changes. Accuracy of ± 0.2 percent was obtained at ± 1 C. It was found that speed of hydration varies with temperature during the first 8 hours; hydration at 150 hours is constant and maximum between 20 and 45 C. This was checked by physical tests.

Manufactured concrete products and the water problem (Les produits en beton manufacture et le probleme de l' eau)

J. P. LEVY, *Revue des Materiaux* (Paris), No. 420, Sept. 1950, pp. 285-297; No. 421, Oct. 1950, pp. 319-322
Reviewed by PHILLIP L. MELVILLE

This study covers the properties and behavior of mixing water as well as free, absorbed and surface moistures. The work of Abrams is reviewed. The use of various water-cement ratios is discussed as a function of various molds. American methods for moisture control are recommended. Various standards related to moisture absorption are given or suggested for blocks, tiles, shingles, fences and pipes. The use of waterproofing agents is recommended especially if added to the mix.

Cylindrical shells of reinforced concrete, Part I

K. HRUBAN, *Transactions of the Dr. E. Benes' Technical University*, Brno (Czechoslovakia), No. 71, 1950, 48 pp.
AUTHOR'S SUMMARY

From the voluminous theory of shells, this study presents the general definitions of internal forces, their relation to deformations and equations of equilibrium and of compatibility. Furthermore, the internal forces are solved for various types of loading by use of the membrane theory. The basis for the Russian methods of short shells is discussed and an approximate solution of long cylindrical shells is shown for the case of sym-

metry about the longitudinal vertical plane through the crown section.

The application of prestressing to shell roof construction

H. G. Cousins, *The Reinforced Concrete Review* (London), V. II, No. 3, July 1950, pp. 189-199
Reviewed by C. P. Siess

Reference is made principally to the multiple barrel arch roof. In general prestressing is recommended for end frames and edge beams only. However, for shells continuous over the end frames or cantilevered, prestressing the shell itself is suggested. In this case the prestress cables are straight and follow generating lines of the shell.

Construction of the Conflans bridge (La construction du pont de Conflans—Fin—D'Oise)

Legrand and Houdin, *Annales de L'Institut Technique du Batiment et des Travaux Publics* (Paris), Travaux Publics No. 7, No. 152, New Series, Oct. 1950
Reviewed by C. P. Siess

Description of design and construction features of a concrete arch highway bridge of 100-m span across the Oise River in France. An interesting feature of the structure is the use of two abutments at each end arranged in tandem. The arch thrust is applied to the front abutment of mass concrete supported on vertical and inclined Franki piles, and a portion of the thrust is transmitted to the earth-filled cellular type rear abutment through a system of jacks placed between the two abutments.

An elastic theory for the torsional strength of rectangular reinforced concrete beams

Henry J. Cowan, *Magazine of Concrete Research* (London), No. 4, 1950, pp. 3-8
Author's Summary

The theories of Rausch and Andersen for the design of torsional shear reinforcement are examined and a more accurate formula is derived, using the strain energy principle. This formula, like that of Rausch, is based on the customary assumptions of the elastic working stress theory. The compound hyperpolic series resulting from the integration of St. Venant's rectangular torsion function is evaluated and reduced to a single constant. This constant is intermediate between those obtained from the approximate theories of Rausch and Andersen.

Meter to determine air content of fresh concrete

M. A. Craven, *Commonwealth Engineer* (Melbourne), V. 37, No. 12, July 1, 1950, pp. 489-492
Reviewed by C. P. Siess

The air meter described was developed by the author in 1947 while at the University of California. It is similar in principle to the device developed by the Washington Department of Highways, but it embodies several features of design and construction which should be of interest. The manner of use and the principles of operation are explained and test results are given. A meter built in New Zealand for field use is illustrated. The author does not recommend this type of meter for laboratory use since, in his opinion, the accuracy obtained is far less than that or the Klein-Walker type.

Thin-shell structures in Spain (Realisations de voutes minces in Espagne)

E. Torroja, *Annales de L'Institute Technique du Batiment et des Travaux Publics. Beton, Beton Arme*, No. 14, No. 164, New Series, Dec. 1950
Reviewed by C. P. Siess

A number of interesting and unusual thin-shell structures are described, including a dome over a market in Algeciras; the Recoletos building in Madrid, in connection with which tests were made on a one-tenth scale model; and a grandstand at the Hippodrome in Madrid with cantilevered hyperbolic shells. Mention is made of damage to the Recoletos building by bombing. Also included are comments on a shell-type structure consisting of a steel beam grillage in place of concrete, and the use of a counterforted hyperbolic shell as a retaining wall for a reservoir.

A plastic design theory for reinforced and prestressed concrete shell roofs

A. L. L. Baker, *Magazine of Concrete Research* (London), No. 4, 1950, pp. 27-34
Author's Summary

A theory is proposed which provides a simple method for designing shell roofs, including prestressed shells. This method takes into account the effect of the plasticity and cracking of the concrete as the ultimate load is approached. Two possible limits of shear stress distribution across a transverse section are considered and it is shown that the bending moment does not vary greatly between these limits. The theory is intended as a basis for experimental work which is to be done in the near future at Imperial College, London.

Experiences and techniques for waterproofing underground structures (L'Experience et la technique au service de l'etancheite le cuvelage)

A. POIRSON, *Annales de L'Institut Technique du Batiment et des Travaux Publics* (Paris), Equipement Technique No. 10, No. 151, New Series, Oct. 1950
Reviewed by C. P. SIESS

Methods for making underground structures water-tight are discussed. Emphasis is placed on the use of concrete protected bituminous membranes either on the exterior or interior face. Numerous drawings illustrate the various methods described. For exterior application the compression of the membrane is considered and test results are given for the compressibility of 4 mm-thick bitumen and asphalt membranes of various types. Details of placing linings around pipes through a wall, at joints, etc. are also presented.

Application of new techniques to the solution of an efflorescence problem

W. E. BROWNELL, *Journal of the American Ceramic Society*, V. 33, No. 12, Dec. 1, 1950, pp. 360-63
AUTHOR'S SUMMARY

Efflorescence of the products of a particular shale was studied. Sodium sulfate was found to be the major effloresecing salt. This salt was formed during firing by the action of sulfur gases, from the oxidation of pyrite, on sodium minerals in the shale. Additions of silica were effective in reducing efflorescence by reacting with the sodium sulfate under a slightly reducing atmosphere. Ammonium chloride additions destroyed the pyrite at a low temperature, thereby preventing the formation of sodium sulfate. These additions may be used to solve similar efflorescence problems. The methods employed are applicable to all cases of efflorescence.

Design and test of helicoid structures (Contributo allo studio delle strutture elicoidali)

RICCARDO F. BALDACCI, *Giornale Del Genio Civile* (Rome), V. 88, No. 1, Jan. 1950, pp. 23-47
Reviewed by GENNARO MIANULLI

This article reports studies and tests on the elastic behavior of a reinforced concrete helicoid structure, limited to two particular conditions of supports and under different loading assumptions. The experimental results are compared with those obtained by computation, which are corrected with the method of least squares to compensate for the uncertain values of a few dimensions not directly observable. The slight variations between the effective and theoretical displacements of the terminal section confirms the accuracy of the analysis followed. The study of the structure was completed by an investigation of the dynamic behavior under moving loads.

Long-span reinforced concrete girder bridges prestressed with wire ropes and supplementary compound (Weitgespannte Stahlbetonbalkenbruecken mit Vorspannung durch Seile und nachtraeglichem Verbund)

F. DISCHINGER, *Beton-u. Stahlbetonbau* (Berlin), V. 45, No. 5, pp. 97-99
Reviewed by RUDOLPH FISCHL

Professor Dischinger's patented method of prestressing reinforced concrete girders with trussed wire ropes spanned between the girders, has been described in detail in earlier papers. This paper develops a new idea for anchoring the wire ropes more effectively over the interior support of continuous girders.

An example for a continuous girder bridge over three spans, 262-590-262 ft shows the economy of this construction. The average quantities per sq ft of area are 5.1 cu ft reinforced concrete and 15.6 lb wire rope with an ultimate tensile stress of 227,500 psi.

Study of the influence of sheathing on the distribution of moments of statically indeterminate structures (Untersuchungen ueber den Einfluss der Einruestung auf die Momentenverteilung bei statisch unbestimmten Systemen)

M. ZERNER, *Beton-u. Stahlbetonbau* (Berlin), V. 45, No. 5, pp. 118-121
Reviewed by RUDOLPH FISCHL

An example of a two-story rigid frame shows how the partial removal of the sheathing in lower floors influences the values of dead load moments, as designed by conventional methods. The difference between true moments and design moments can be remarkable and grows with the ratio of dead load and live load. The author developed a simple approximate method to compute this influence without much additional work. This method is published in the thesis for his doctor's degree at the Technical University in Hannover, from which this paper is an excerpt.

Testing cement base paints and dampproofers

WILLIAM M. SPURGEON, ASTM *Bulletin*, No. 171, Jan. 1951, pp. 53-57

AUTHOR'S SUMMARY

Tests made on 23 commercial cement paints, 21 of which are used for dampproofing masonry walls, show that they vary widely in porosity (water absorptions range from 2.4 to 65.1 percent by weight) and 7-day compressive strengths (0 to 3300 psi). In general, for comparable curing conditions, high absorption values accompanied low compressive strengths and vice versa.

The theory of dampproofing is discussed briefly, and the need for further development of a simple, short screening test to supplement weathering data for cement-base dampproofers is shown. The need for establishing specifications for these materials is also pointed out.

Bibliography on management of construction jobs

The Engineering Societies Library, New York. $2.00

To make papers directly pertinent to construction job organization and management easily and quickly available, the Engineering Societies Library has compiled an annotated bibliography of 52 selected references to books and periodical articles dealing with construction job planning, work scheduling, cost estimating, organization, preparation of material-requirement charts, material-delivery time scheduling, job progress records, time studies, material and equipment control, and contracting procedures on various kinds of contracting and building jobs such as small and large-scale housing, office and factory building, tunnel, highway and dam construction.

Cement clinker seen in a microscope under reflected light (Le clinker de ciment artificial vu au microscope en lumiere reflechie)

P. LHOPITALLIER and L. M. JAMES, *Revue des Materiaux* (Paris), Special Issue, Oct. 1950, pp. 17-28

Reviewed by PHILLIP L. MELVILLE

The clinkers were specially treated by impregnation with a synthetic resin or by polishing with abrasives. Components of the clinker were identified with nitric acid, borax, fluorhydric acid, sodium hydroxide, water. Crystals were identified by microscopic study. The data was used to control manufacture. In particular the extent of the unslaked lime is related to the fineness of raw materials; the pore sizes in the clinker to the grain sizes of the raw materials. Cinder from very impure fuels has been observed in the clinker especially when made by the wet process. This does not happen with clean fuels. C_2S is stable only between 1250 and 1900 C.

Investigation of the composition and properties of porous concretes (Etude sur la composition et les proprietes des betons caverneux)

A. BIREBENT, *Annales de L'Institut Technique du Batiment et des Travaux Publics, Beton, Beton Arme*, No. 15, No. 165, New Series, Dec. 1950

Reviewed by C. P. SIESS

Two series of tests are reported. In the first, the density of the concrete was determined for a large variety of mixtures in which the aggregate gradations, and cement and water contents were varied. Gradations were varied according to the equation

$$T = 100\left(\frac{d}{D}\right)^x \text{ (in percent)}$$

for x varying from zero to infinity. Cement contents were 0, 5, 10 and 15 percent by weight of dry materials, and water content was varied for each cement content. From the mixtures thus tested a number were selected and tested further to determine flexural and compressive strengths, modulus of elasticity, and coefficient of thermal conductivity.

About driving on concrete roads (Om at koere paa cementbetonvejene)

E. BOESGAARD, *Beton-Teknik* (Copenhagen), V. 16, No. 4, 1950, pp. 125-130

AUTHOR'S SUMMARY

In this article Eric Boesgaard, editor of the Danish periodical *Motor* reviews the question of concrete roads from a motorist's point of view. He feels sure that a Gallup investigation among Danish road users would prove that the preferred pavement for roads is concrete. He himself has only one reservation: that the concrete road must be well made. Concrete roads are preferred because of (1) the safety one feels when driving on these roads—skidding is practically impossible; (2) the light color of concrete roads; (3) their evenness and (4) the fact that they have a long life and require practically no maintenance.

Mr. Boesgaard put the question to a heavy-truck driver, Mr. Frederiksen. To him also concrete roads seem the best—for safety and comfort, especially in rain and after dark—and for minimum wear and tear on his truck.

Two new concrete roads (To nye betonveje)

P. B. Helwigh, *Beton-Teknik* (Copenhagen), V. 16, No. 4, 1950, pp. 119-124

Author's Summary

The article describes some new methods of road construction used on two concrete roads which carry heavy traffic in the neighborhood of Copenhagen. The specifications for these roads are cited.

Air entrainment was used with good results, but no reduction in W/C was obtained, this being about 0.5. The uniformity of the quality of the concrete was tested by the sound-velocity method. The velocity was about 4.1 to 4.8 km per sec.

The joints were made in accordance with American practice, *viz*, with dummy-joints at 5 m intervals and expansion joints where each day's work ended (about every 100 m). Some of the dummy-joints were made 3 mm wide, and filled with asphalt, others with preformed filling material (3 x 50 mm) to the level of the concrete surface. A vibrator used for finishing the joints is described.

Handbook of chemistry and physics

32nd Edition, 1950-51, Chemical Rubber Publishing Co., Cleveland. 2880 pp.

Since the first edition in 1914 this standard data book has been continually revised and kept up to date. The latest edition contains about 335 pages of completely revised and wholly new material. Changes and additions have been made to 155 pages. Complete revision and enlargement of the table of isotopes reflects rapid developments in nuclear physics. The collection of data and references to the literature now covers 92 pages.

Major revisions have been made in description of the elements, periodic table, properties of commercial plastics, organic analytical reagents, potentials of electrochemical reactions, properties of the amino acids, surface tension, vapor pressure, dielectric constants and plate and film speeds.

Important new tables have been added on steriod hormones, electron work functions of the elements, Centrigrade-Kelvin conversion table and scientific and engineering abbreviations and symbols.

Concrete roads viewed technically and economically (Betonveje fra et teknisk ogoe konomisk synspunkt)

S. Ellert, *Beton-Teknik* (Copenhagen), V. 16, No. 4, 1950, pp. 110-118

Author's Summary

The author points out that highways carrying heavy traffic should be constructed of concrete in preference to bituminous carpet on a foundation of macadam, which has been treated with asphalt emulsion.

This is because in color, carrying capacity, maintenance and durability, concrete is superior to bituminous carpet, and at the same time is of sufficient evenness and roughness to assure pleasant and safe driving. Further, the cost in foreign exchange is lower for concrete roads.

Concrete without reinforcement may be used and the joints must be dummy-joints at short intervals (5 m) with expansion joints at greater intervals (50-100 m).

The subsoil must be carefully examined and if necessary stabilized.

Strict demands must be made as to evenness during construction.

German prestressed concrete railroad ties (Traviesas de hormigon pretensado en los ferrocarriles Alemanes)

Herman Meier, *Publication* No. 97, Instituto Tecnico de la Construccion y del Cemento, Madrid. 36 pp.

Practical conclusions are drawn from detailed tests of wood ties and applied to the design of similar members made of prestressed concrete.

The principles and procedures in mass-producing prestressed concrete ties and their installation in the road bed are described and illustrated. Uniform resilience of the railroad ballast was found essential to successful use of the ties and led to the development of special machines to achieve this condition.

The prestressed ties made it possible to weld the rails into continuous lengths of several kilometers with a consequent reduction in cost of maintenance of way and

of rolling stock. Other advantages are high electrical resistance, ease of manufacture and long life, which is estimated to be 60 years. A comparison of annual costs reflected a substantial saving when prestressed ties were used.

Torsion in reinforced concrete structures (Sulle sollicitazioni di torsione nelle costruzioni in cemento armato)

E. Castiglia, *Il Cemento* (Milan), V. 67, No. 5, May 1950, pp. 85-88

Reviewed by Gennaro Manulli

This investigation consists of determining the effect of torsion in restrained beams stressed by the action of a twisting couple combined with a uniformly distributed load.

Two theorems are established by observing the similarity between a beam fixed at both ends, with stresses induced by a combined uniform distributed load and a twisting couple, and a simply supported beam of equal span and equal cross section supporting a combined uniform distributed load and a set of concentrated loads which substitute the couples in number and proportionally. Diagrams are plotted for each condition of loading, for torsional moment, for angle of twist and for shear and bending moment. The results obtained from this similarity are applied to beams receiving their loads from one side only, and restrained against rotation. Cantilevers, spandrel beams and beams around openings of stairs and elevators are typical examples.

Elementary theory and design of flexural members

Jamison Vawter and James G. Clark, John Wiley & Sons, Inc., New York, 1950, 215 pp. $4.00

Reviewed by A. A. Topractsoglou

This well-written book is a suitable textbook for a one semester course on theory and design for non-civil engineers. The book covers the basic theory of flexure and applies the theory to the design of members which carry bending moments.

In the first two chapters the theory of bending for homogeneous and non-homogeneous beams is presented. In the chapters that follow, the theory is applied to the design of steel beams and girders. The design of reinforced concrete beams and columns is presented in Chapter 5. Bending in unsymmetrical sections and bending in special beams, including those with thin webs, is discussed in Chapters 6 and 7.

The different theories of flexure are presented clearly and in an elementary way. Throughout the book, the authors show that the same basic laws of statics and strength of materials govern the design of flexural members, no matter what structural material is used.

A great number of illustrative examples are presented in each chapter to show the application of theory. Adequate number of problems are given at the end of each chapter for assignment to students. In general the book lends itself readily to teaching.

Moment distribution analysis of two-span arched frames with elastic pier

Thomas P. Revelise, *Public Roads*, V. 26, No. 4, Oct. 1950, pp. 65-84

Author's Summary

During recent years multiple arches in continuous series have assumed considerable importance in the structural engineering field, and several methods of analysis of such structures by moment and thrust distribution have been developed or proposed. This article presents an adaptation of the method of moment distribution to the analysis of two-span arched frames with elastic center pier and with either fixed or hinged footings. The method is applicable to any two-span continuous arch, but is arranged particularly for convenience in the analysis of arched frames because of the increasingly frequent use of this type of structure for grade separations for divided highways.

Part I is devoted to the necessary mathematical development for a structure with hinged footings, a discussion of procedure, and a sample analysis of an unsymmetrical two-span frame with hinged footings. In Part II expressions for a structure with fixed footings are developed, followed by a discussion of procedure and a sample analysis of the same structure as that used in Part I, but with footings fixed.

The use of forms for tabulating computations makes most of the analysis procedure a mechanical operation by which results can be obtained rapidly and accurately by designers of limited experience.

A method of design for shell concrete roofs using prestressed edge beams

V. M. SILVERA, *Magazine of Concrete Research* (London), No. 4, 1950, pp. 9-14

AUTHOR'S SUMMARY

In a beam provided with prestressing cables which follow a continuous curve between supports, the prestressing force resolves itself into two components: a horizontal compressive force which remains nearly constant throughout the span and a vertical distributed load acting upward and balanced by two downward reactions at the anchorages over the supports.

By selecting a suitable curve for the cable, this vertical distributed load can be made uniform throughout the span; it can also be made to equal in value the uniformly distributed permanent loads acting on the beam, thereby cancelling all deflections caused by such loads.

Prestressing forces are usually designed to counteract tensile stresses, the effect of such forces on the deflection being purely incidental. However, in thin concrete shells the vertical prestressing forces provide an effective means of cancelling the edge disturbances caused by deflections in edge beams, thereby enabling the thin shells to be regarded as membranes without any appreciable bending stresses.

An attempt is made to analyze the effect on thin shell structures of the prestressing forces resulting from suitable arrangement of cables. Some reference is also made to polygonal shell structures.

Nomograms for concrete design (Nomogrammen voor betonconstructies)

H. W. F. C. HEMAN, *Cement* (Amsterdam), No. 23-24, 1950, pp. 536-549

Reviewed by J. W. T. VAN ERP

It is surprising that nomographic charts never have become popular for concrete design. This can only be because they are relatively unknown to the structural designer or because there is no literature on them, or both. Anyone at all familiar with them knows that they are a very practical and efficient tool for many purposes as is proved by their popularity in many other branches of science and technology.

This paper demonstrates their practicability as a tool for everyday design work. Their chief merit is an immediate (graphic) solution for equations with three, four, or more unknowns. Such equations are those governing reinforced concrete design. In this country it has become universal practice to solve them by means of tables and simple arithmetic and only when tables become too complex have diagrams become popular, like those for eccentrically loaded columns, adopted in the ACI *Reinforced Concrete Handbook*. In a nomogram, however, it is possible to present much more complex relations, involving many variables and still embody them in an easily readable chart.

Their advantages are (1) quick reading, (2) no chance of calculation errors, (3) an accuracy equal to that of the 10-in. slide rule (folded scales) for an 8 x 10-in. chart, (4) no special knowledge required to manipulate them, except an ability to read logarithmic scales as on a slide rule, (5) simple and accurate interpolation like on a slide rule, where it is an everyday routine matter. Compare this with the clumsy and none too accurate operation like interpolation of a chart on cartesian coordinates like the eccentric column loading diagrams in the ACI handbook and (6) the designer gets a clear insight of what he is doing, much more so than in the blind manipulation of tables.

A nomogram will give the required effective depths and steel area at various concrete stresses for a given bending moment. The designer sees exactly how these quantities vary in relation to each other. Cost lines (based on given unit costs) may be incorporated so that the designer can select quantities that lead to minimum over-all cost. This is an added advantage not even possible with conventional methods.

The article gives only a limited number of charts on flexural members and eccentrically loaded columns, but these few examples eloquently demonstrate their possibilities. They can also be used in stress investigation, giving instant readings. Anyone familiar with the tedious process of conventional stress investigation will fully appreciate this.

This reviewer is convinced that nomograms should be adopted wholesale by the practicing designer as a standard design method. That factors of economy can be definitely incorporated in them makes it even more opportune in these days than ever.